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PREFACE

The International Library of Technology is the outgrowth of a large and increasing demand that has arisen for the Reference Libraries of the International Correspondence Schools on the part of those who are not students of the Schools. As the volumes composing this Library are all printed from the same plates used in printing the Reference Libraries above mentioned, a few words are necessary regarding the scope and purpose of the instruction imparted to the students of—and the class of students taught by—these Schools, in order to afford a clear understanding of their salient and unique features.

The only requirement for admission to any of the courses offered by the International Correspondence Schools, is that the applicant shall be able to read the English language and to write it sufficiently well to make his written answers to the questions asked him intelligible. Each course is complete in itself, and no textbooks are required other than those prepared by the Schools for the particular course selected. The students themselves are from every class, trade, and profession and from every country; they are, almost without exception, busily engaged in some vocation, and can spare but little time for study, and that usually outside of their regular working hours. The information desired is such as can be immediately applied in practice, so that the student may be enabled to exchange his present vocation for a more congenial one, or to rise to a higher level in the one he now pursues. Furthermore, he wishes to ·ain a good working knowledge of the subjects treated in ·hortest time and in the most direct manner possible.

In meeting these requirements, we have produced a set of books that in many respects, and particularly in the general plan followed, are absolutely unique. In the majority of subjects treated the knowledge of mathematics required is limited to the simplest principles of arithmetic and mensuration, and in no case is any greater knowledge of mathematics needed than the simplest elementary principles of algebra, geometry, and trigonometry, with a thorough, practical acquaintance with the use of the logarithmic table. To effect this result, derivations of rules and formulas are omitted, but thorough and complete instructions are given regarding how, when, and under what circumstances any particular rule, formula, or process should be applied; and whenever possible one or more examples, such as would be likely to arise in actual practice—together with their solutions—are given to illustrate and explain its application.

In preparing these textbooks, it has been our constant endeavor to view the matter from the student's standpoint, and to try and anticipate everything that would cause him trouble. The utmost pains have been taken to avoid and correct any and all ambiguous expressions—both those due to faulty rhetoric and those due to insufficiency of statement or explanation. As the best way to make a statement, explanation, or description clear is to give a picture or a diagram in connection with it, illustrations have been used almost without limit. The illustrations have in all cases

indexes are so full and complete, that it can at once be made available to the reader. The numerous examples and explanatory remarks, together with the absence of long demonstrations and abstruse mathematical calculations, are of great assistance in helping one to select the proper formula, method, or process and in teaching him how and when it should be used.

Two of the volumes composing this library, of which this is the second, are devoted to the subject of structural engineering, and may, in the main, be considered as illustrating the practical application of the information given in the first volume, in which the elements of a structure were considered in detail. The present volume deals with the combination of these elements into various structures; it considers the foundation of a building; the walls, constructed of various materials; the openings in the walls, such as doors and windows, to which are applied the theory of arches; and, finally, the roof, constructed either of wood or iron. Considerable attention is paid to the important subject of wind pressure.

Another matter of great importance is Fireproofing, which has been fully treated. The volume closes with a paper on Specifications, a subject of vital importance to the engineer, as well as to the bidder, a well-drawn specification being a guarantee of good work and a protection in case of dispute.

The method of numbering the pages, cuts, articles, etc. is such that each subject or part, when the subject is divided into two or more parts, is complete in itself; hence, in order to make the index intelligible, it was necessary to give each subject or part a number. This number is placed at the top of each page, on the headline, opposite the page number; and to distinguish it from the page number it is preceded by the printer's section mark (§). Consequently, a reference such as § 16, page 26, will be readily found by looking along the inside edges of the headlines until § 16 is found, and then through § 16 until page 26 is found.

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STATICS OF MASONRY

(PART 1)

INTRODUCTION

1. Masonry construction may be classified according to the methods of assembling the material, in which case the primary heads will be *single stones and monoliths*, *walls and piers*, *arches*, *vaults*, *and domes*. As in building construction all these elements are encountered, each must be considered for *general design*, *stability*, and *execution*.

This Section is to be particularly devoted to the consideration of the stability of masonry and the features of general design relating to the strength and durability of the construction.

2. General Design.—Before the question of stability can be determined, the **general design** must be studied and the approximate lines of the structure laid down, then the weights may be figured, the pressures calculated, and the effects of the accidental loads analyzed, after which the sizes of the structural elements of the design may be increased or diminished and the weights redistributed, so that greater stability is secured or further economy of design obtained.

The general design of masonry in building construction belongs properly to the province of the architect or structural engineer. The engineer's knowledge is, however, requisite when the construction is of an unusual character and when the strength of the structure is of primary importance. In iron and steel construction, designing by precedent, that is, from existing work as examples, is poor

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practice, for there are exact formulas and rules by which the engineer is governed. In masonry construction, however, it is different, for there are so many unknown factors that it is impossible to work out the design by fixed rules; therefore, it is not uncommon practice to map out the general design and then carefully compare it with examples of existing work that are subjected to practically the same conditions. After this comparison has been made, the design should be examined and carefully analyzed by the methods and laws of practice that have been agreed on and generally adopted by the best engineers. The stability of small arches, lintels, and unimportant details is generally sufficiently determined by inspection, for where the workmanship and materials are of good quality, such features usually possess a sufficiently high factor of safety to warrant this practice. But the important elements of the construction, that is, those elements on which, possibly, the stability of the entire structure will depend, must be as carefully designed as principal columns, girders, or framed structures.

3. Stability.—In this quality of masonry construction, the engineer is particularly interested, and it constitutes the first element of his training. The term *stability*, when applied to masonry construction, implies that the mass of material is so disposed that the structure is in equilibrium and secure against failure from the action of gravity exerted vertically, or when conveyed by arches, vaults, or domes, horizontally or obliquely, and from the action of wind pressure. **Stability**, besides being dependent on the disposition of the material, is not independent of its strength, for, even though the mass of the material were so arranged as to be in stable equilibrium, if the units of which it was composed were of insufficient strength, the disposition of the material would be changed by crushing or breaking, and the destruction of the work hastened.

The *first requirement* for stability involves the equilibrium of the structure, and in general the conditions that govern equilibrium are the following:

1. The center of gravity of the mass should not fall without the base of the structure.

2. The overturning moment produced by wind pressure, or the thrust of some extraneous structure, should be sufficiently resisted by the moment of the weight of the mass about the point of rotation.

3. The lines of pressure instituted in masonry construction, such as walls, arches, and domes, must be so arranged that the forces they represent do not tend to produce tension in the joints between the material units of which the mass is composed.

The *second requirement* of stability relates to the strength of the material of which the structure is composed; it dictates that:

1. The safe crushing strength of the stone must not be exceeded.

2. The safe compressive strength of the mortar must be such as to insure it against squeezing out of the joint, thus causing undue settlement.

3. The bending moments produced in the stone should not be greater than the safe transverse strength of the material.

4. The shearing action of the load should be investigated and provided for.

5. If tension exists in the structure, its amount should be well within the safe tensile strength of the stone or mortar, as the case may be.

4. Execution or Production.—The execution or production of masonry is treated in this Section only so far as it relates to the principles of stability. It does not include the investigation of the nature and manufacture of such materials as brick, cement, concrete, and terra cotta, and the quarrying and shaping of stone, as well as the processes of setting and laying, of which arts and trades the architect and structural engineer must have an accurate general knowledge; the information necessary along these lines is given in other Sections of the Course.

SINGLE STONES AND MONOLITHS

5. Single stones and **monoliths** of structural importance in building construction include *bearing blocks*, *lintels*, *flagstones*, *columns*, and *corbels*. With the exception of the architectural treatment of the exposed edges or surfaces of the stones, which is decided by the design of the architect, the general form and dimensions are determined by the strength required. The depth, or thickness, of any stone of structural importance is usually determined by the shearing and bending stresses, while the area of its bed, which is governed by its length and width, is determined by the pressure on the joint.

BEARING BLOCKS

6. Bearing blocks are used under the foot of columns or posts and under the ends of beams, girders, or trusses. When stone bearing blocks are used under columns or posts, they are called **capstones**; when used in walls or on the tops of pilasters as the supports for the ends of beams, girders, or roof trusses, they are called **templets**. The purpose of stone bearing blocks is to transmit the load from the structural member, as the column, beam, or girder, to the masonry. The bearing blocks accomplish this object by enlarging the bearing area sufficiently to reduce the unit pressure on the weaker material, which unit pressure must not exceed the allowable unit bearing strength of the underlying masonry.

It is not uncommon to enlarge the bearing area of walls and piers on the soil, by a course of masonry or concrete that projects beyond the wall or pier line. These projecting courses are called **footings**, and are frequently made a single stone in width, or more commonly of concrete, which, when it has set, partakes of the nature of a single stone. The projecting portion of a footing course beneath a pier or

column is similar in its action, with reference to the load and support, to an inverted cantilever. The footing course, when regarded in this manner, is supported at the edge of the pier or wall and the load is the pressure of the soil from beneath.

The unit of measurement adopted in proportioning bearing blocks for direct stresses is *pounds per square inch*, while the unit of measurement used in proportioning the area of foundation footings is *pounds per square foot*.

PROPORTIONING THE BEARING AREA

7. The center of gravity of all the loads acting on a bearing block is called the **center of pressure**. This pressure must be equaled by the reaction or pressure from beneath, which, too, may be considered as applied at a certain point, which is designated by the term **center of resistance**. In order that the bearing block may be held

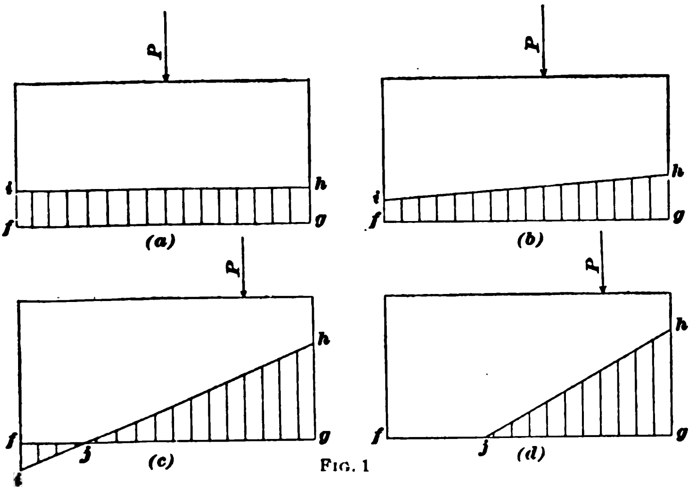


FIG. 1

in equilibrium, the center of resistance must be directly under the center of pressure.

When the center of pressure is applied at the center of gravity of the area of the joint, the unit pressure on the joint will be uniform, in which case the intensity of pressure per lineal inch is equal to the load divided by the length of

the footing, in inches. This is graphically represented in Fig. 1 (*a*), where P equals the resultant of the load, while the pressure beneath the footing may be designated by the ordinates included in the rectangle $fghi$. The height if, or the length of the ordinates of the rectangle, is obtained by dividing the pressure from the load P by the length of the line fg, which is the length of the joint. In this case the base may be supposed to consist of 16 lineal inches. If the pressure P is equal to 16,000 pounds, the pressure per lineal inch would be $\frac{16,000}{16}$ = 1,000 pounds. If the unit length of an ordinate is 1 inch, representing a pressure of 500 pounds, the height of if should be 2 inches, drawn to the same scale as the base fg.

8. Frequently the center of pressure does not coincide with the center of gravity of the bearing area of the block, in which case the pressure on the joint will not be uniform, but will vary from one end of the bearing area to the other, as is graphically shown by the trapezoid $fghi$, Fig. 1 (*b*). The unit pressure at if and hg, when obtained, may be used to find the unit pressure at any point.

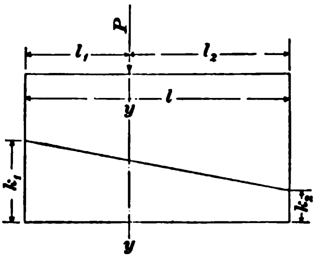


FIG. 2

This is done by laying off the ordinates if and hg to some scale of pounds to lineal measurement and completing the trapezoid; then the pressure at any point throughout the bed joint of the bearing block may be determined by measuring the ordinate at that point.

In order to obtain the maximum and minimum pressures per unit of lineal measurement, the following formulas are employed; the various values will be understood by reference to Fig. 2.

$$k_1 = \frac{2P(2l - 3l_1)}{l^2} \qquad (1)$$

$$k_2 = \frac{2P(2l - 3l_2)}{l^2} \qquad (2)$$

in which k_1 = pressure, per lineal inch, at the edge nearest to the center of pressure;

k_2 = pressure, per lineal inch, at the edge farthest from the center of pressure;

l = length of the stone, in inches;

l_1 = distance from the center of pressure to the nearest edge of the bearing stone;

l_2 = distance from the center of pressure to the farthest edge of the bearing stone;

P = vertical load applied at the center of pressure.

The following article describes the derivation of formulas **1** and **2** and, although of secondary importance, it may be interesting to know how these formulas are derived.

9. Formulas **1** and **2** are readily derived when it is known that the distances from the ends of a trapezoid to a line passing through the center of gravity of its surface are equal to l_1 and l_2, as determined by the following formulas:

$$l_1 = \frac{(k_1 + 2k_2)l}{(k_1 + k_2)3} \qquad \text{(a)}$$

$$l_2 = \frac{(k_2 + 2k_1)l}{(k_1 + k_2)3} \qquad \text{(b)}$$

The values of l_1, l_2, k_1, and k_2 are clear by reference to Fig. 2, but here k_1 and k_2, instead of referring directly to pressure, are considered as the heights of the parallel sides of the trapezoid, since the trapezoidal figure represents accurately the variation of pressure.

The condition that must obtain in order that the bearing block may be in equilibrium is that the axis yy, which passes through the center of gravity of the area of the trapezoid, must coincide with the center of pressure represented by P and that the upward reactions must equal the downward pressure. From this last condition, the value of P may be expressed in terms of k_1, k_2, and l, thus,

$$P = \frac{(k_1 + k_2)l}{2} \qquad \text{(c)}$$

so that

$$k_1 = \frac{2P - lk_2}{l} \qquad \text{(d)}$$

and

$$k_2 = \frac{2P - lk_1}{l} \qquad \text{(e)}$$

Formulas **a** and **b** for obtaining the values of l_1 and l_2 may be simplified by clearing of fractions, when they respectively take the following form:

$$3l_1k_1 + 3l_1k_2 = lk_1 + 2lk_2$$

$$3l_2k_1 + 3l_2k_2 = lk_2 + 2lk_1$$

which, when still further simplified, become

$$k_1(3\,l_1 - l) = k_2(2\,l - 3\,l_1)$$
$$k_2(3\,l_2 - l) = k_1(2\,l - 3\,l_2)$$

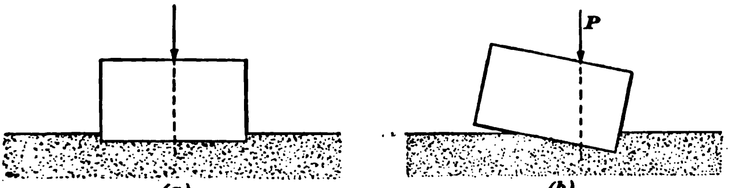
In each of these last equations, the values of k_1 and k_2 given in formulas **d** and **e** may be substituted, so that the equations are changed to the following forms, which, when simplified as shown, give formulas **1** and **2**:

$$k_1(3\,l_1 - l) = \frac{2\,P - l\,k_1}{l}(2\,l - 3\,l_1)$$
$$k_1(3\,l_1\,l - l^2) = 4\,P\,l - 2\,k_1\,l^2 - 6\,l_1\,P + 3\,l\,l_1\,k_1$$
$$3\,l\,l_1\,k_1 - k_1\,l^2 + 2\,k_1\,l^2 - 3\,l\,l_1\,k_1 = 4\,P\,l - 6\,l_1\,P$$
$$k_1\,l^2 = 4\,P\,l - 6\,l_1\,P$$
$$k_1 = \frac{2\,P(2\,l - 3\,l_1)}{l^2}$$
$$k_2(3\,l_2 - l) = \frac{2\,P - l\,k_2}{l}(2\,l - 3\,l_2)$$
$$k_2(3\,l_2\,l - l^2) = 4\,P\,l - 2\,k_2\,l^2 - 6\,P\,l_2 + 3\,k_2\,l_2\,l$$
$$3\,k_2\,l_2\,l - k_2\,l^2 - 3\,k_2\,l_2\,l + 2\,k_2\,l^2 = 4\,P\,l - 6\,P\,l_2$$
$$k_2\,l^2 = 4\,P\,l - 6\,P\,l_2$$
$$k_2 = \frac{2\,P(2\,l - 3\,l_2)}{l^2}$$

10. Referring again to Fig. 2, it will be observed that as the length l_1 decreases, the value of k_1 will increase, and it may be determined from formula **2** that the intensity of the pressure k_2 at the farthest edge will equal zero when l_1 is equal to one-third of the distance l. This is evident when it is remembered that the center of gravity of a triangle is at a distance of one-third its height from the base and that when the distance k_2, Fig. 2, is zero, the figure representing the pressure on the bed becomes a triangle. When the center of pressure P approaches the end of the bearing block nearer than one-third the length of the block, tension is created at the farthest edge of the joint and stresses are produced on the bed of the bearing stone, as illustrated in Fig. 1 (c). In the case here shown, the center of pressure P is considerably nearer the right-hand edge than one-third the length of the bearing stone, which causes the direct compression at the right-hand edge of the bearing block to be considerably increased, and produces tension at and adjacent to the left-hand edge of the joint, which is diagrammatically represented by the triangle $f\,i\,j$.

11. In proportioning blocks for bearing on the bed, the center of pressure should never be located without the middle third when it can be avoided; where it is unavoidable, the tensile resistance due to the adhesiveness of the mortar at the joint must be neglected and the block regarded as resting on some material, such as sand, which is capable of resisting compression only.

The condition that prevails when the bearing block is supported on a semiyielding material incapable of exerting tensile resistance is shown in Fig. 3. At (*a*) is shown a bearing block in which the center of pressure is located at the center of the bearing area, while at (*b*) is designated a block on which the center of pressure has moved toward the right, causing the right-hand edge of the stone to sink more



(*a*) FIG. 3 (*b*)

than the left-hand edge until the distance of P from the right-hand edge becomes less than one-third the length of the stone, after which the left-hand edge of the stone rises above the bearing, and the stone takes the position shown. The conditions that exist in Fig. 3 (*b*) produce pressures on the bed of the stone that are diagrammatically shown in Fig. 1 (*d*).

Should the center of pressure fall without the middle third, the maximum pressure k_1, per lineal inch, at the outside edge, may be determined by the formula

$$k_1 = \frac{2\,P}{3\,l_1} \qquad (3)$$

in which P = total load on the bearing block;

l_1 = distance from the center of pressure to the nearest edge of the block.

If the center of pressure P falls within the middle third of the length of the bearing block, the maximum pressure,

or k_1, may be calculated by formula **1.** However, if the center of pressure falls on the outside edge of the portion of the block included within the middle third of its length, the amount of the maximum intensity of pressure at the outside edge is immediately determined, for under this condition the maximum intensity of pressure is equal to twice the intensity of pressure that would exist if the center of pressure coincided with the center of the bearing area. For instance, when $l_1 = \frac{l}{3}$, by substitution in formula **1,**

$$k_1 = 2P\left(\frac{2l - \frac{3l}{3}}{l^2}\right)$$

or $$k_1 = \frac{2Pl}{l^2},$$

hence $$k_1 = \frac{2P}{l}, \qquad (4)$$

which is twice the mean intensity of pressure per lineal inch when the center of pressure is at the center of gravity of the bearing area, for in the latter case the intensity of pressure per unit of lineal measurement is equal to $\frac{P}{l}$.

12. A bearing block on which the pressure is applied at the center of the bearing area is readily proportioned by first determining the allowable unit bearing value of the masonry, which is governed by the character of the work and the nature of the mortar. When the allowable unit bearing value has been decided on, the required area of the stone on the bed may be found by dividing the load or pressure by the safe unit bearing pressure, and the dimensions of the stone may then be proportioned.

If the center of the load, or pressure, on the block does not correspond in position with the center of the bearing area, the length and width of the block may be assumed, and formulas **1** and **2** applied in order to find the maximum intensity of pressure at the edge of the block. When the maximum unit pressure exceeds the allowable unit bearing

value of the masonry, the amount or position of the load must be altered or the block redimensioned and formulas **1** and **2** again applied.

13. These principles may be applied conveniently, in practice, for the proportioning of bearing stones, the following examples illustrating their application and the method of procedure:

EXAMPLE.—A 20-inch I beam, bearing on an 18-inch rubble wall, laid in Rosendale cement mortar, sustains a load of 40,000 pounds. What size bearing block of bluestone should be used?

SOLUTION.—Since the material offering the least resistance to crushing is the cement mortar of the joint, the safe bearing resistance is taken at 150 lb. per sq. in. Therefore, the required bearing area of the block, or templet, on the wall is equal to 40,000 ÷ 150, or 267 sq. in. The width of the stone will be the thickness of the wall when the best stones are used on the edge, and the face of the wall is laid up plumb and true. In this case, however, it is assumed that the wall is of ordinary construction with irregular stones at the edges, so that 16 in. will be considered as the actual bearing width of the stone, as indicated in Fig. 4.

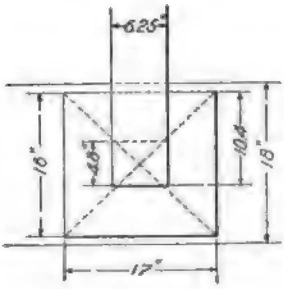


FIG. 4

If the required bearing area of the block, or templet, is 267 sq. in. and the assumed bearing width of the block is 16 in., the length of the block on its bed will equal 267 ÷ 16, or 16.69 in., say 17 in. A bearing block must, therefore, be provided at least 16 in. wide by 17 in. long, as shown in Fig. 4. Ans.

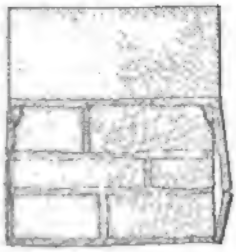


FIG. 5

14. Where the bearing block approaches the thickness of the rubble wall, it is customary to make it equal in width to the thickness of the wall, and when the latter is to be plastered the bearing block is projected somewhat beyond the face of the wall. In all cases, however, the theoretical width of the block, and not its actual width, must be considered in determining the other dimensions; for the plaster and stone tend to spall, or chip off,

as shown in Fig. 5, so that the best practice is to assume the outside edge of the stone as being several inches from the edge of the wall.

EXAMPLE.—In the preceding example, the load of 40,000 pounds was transmitted to the bearing block through a 20-inch I beam having a flange width of $6\frac{1}{4}$ inches. Providing the bearing block is of bluestone, what size steel plate will be required under the flange so that the beam will bear centrally on the stone?

SOLUTION.—The plate should have very little projection beyond the edges of the flange so as to avoid bending moment on the plate. The allowable unit bearing value of a bluestone, capstone, or templet is 500 lb. per sq. in., hence, if the plate is made 8 in. wide the pressure per lineal inch will equal 4,000 lb. The total pressure on the plate is 40,000 lb.; hence, the length of the bearing plate equals $40,000 \div 4,000 = 10$ in. Ans.

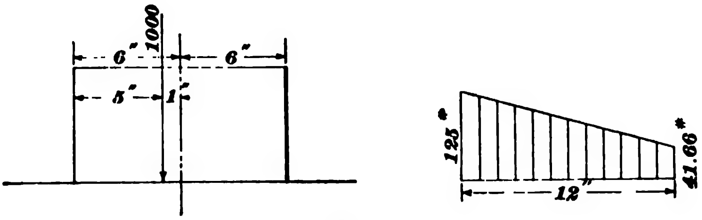


FIG. 6

15. In practice, it is not unusual to place the beam or girder directly on the stone templet, but where the load is heavy, it is better construction to use the bearing plate, placing it centrally with respect to the templet, in order to

The values of k_1 and k_2, equal, respectively, to 125 and 41.66 lb., represent the maximum and minimum intensity of pressure, in pounds per lineal inch, at the extreme edges of the block. The width of the block is 10 in., therefore the maximum unit pressure equals 125 ÷ 10, or 12.5 lb., while the minimum unit pressure equals 41.66 ÷ 10, or 4.166 lb. Ans.

From the results of this example it will be observed that the maximum unit pressure of 125 pounds is well within the safe unit bearing value of brickwork or good stonework laid up in Rosendale cement mortar. If the maximum unit pressure, as calculated in the example, had exceeded the allowable unit bearing value of the underlying masonry, the original dimensions of the bearing block would have to be altered and the maximum pressure again calculated to find whether it would come within the safe unit bearing value.

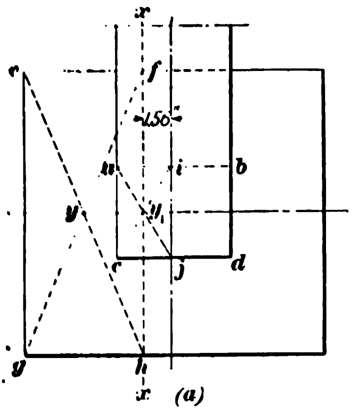
PROPORTIONING FOR BENDING MOMENT

16. Block Centrally Loaded.—A bearing stone, besides being proportioned for bearing on its bed, must be analyzed for transverse stress when the transmitting area of the superimposed load is less than the bearing area of the block; that is, a bending moment is created when the area acted on by the reaction from the wall is larger than the bearing area of the column, beam, girder, or truss.

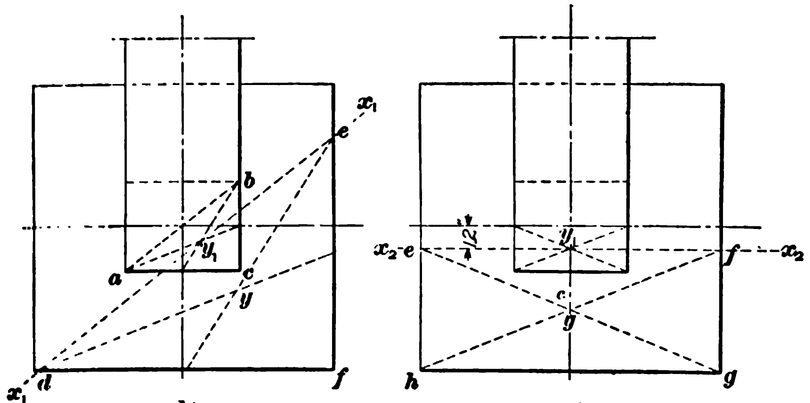
Where the bearing block occupies the position of a templet, and is built solidly into the wall, it is not subjected to as great a bending moment as it would if it were a capstone or were located at the top of the wall. This is due to the fact that the superimposed masonry on the ends of the bearing stone, or on that portion of the stone not covered by the bearing plate of the beam, girder, or column, assists in resisting the upward pressure due to the reaction from the masonry beneath. Nevertheless, it is well to proportion all such bearing stones as though their ends were free, for this condition may practically exist from settlement or shrinkage.

The maximum bending moment is produced on a bearing block when a maximum portion of the bearing area of the bed of the block has its center of gravity projecting

a maximum distance beyond the center of gravity of one-half the bearing area of the superimposed load. For instance, in Fig. 7 (*a*), (*b*), and (*c*) are shown plan views of a bearing block, or templet, with the flange of an I beam resting on a bearing plate designated by the rectangle *a b d c*, Fig. 7 (*a*). It is necessary to find the maximum bending moment on the block, and as this may exist about any of the axes $x\ x$, $x_1\ x_1$, or $x_2\ x_2$ of the plans (*a*), (*b*), and (*c*), the bending moment about these several axes must be analyzed. In Fig. 7 (*a*), the axis $x\ x$, which is assumed to be the line of fracture of the block, is drawn through the center of gravity of one-half the bearing area of the plate *a b d c*, or of the rectangle *a i j c*. As will be observed from the figure, the center of gravity of the rectangle is determined by the intersection of the diagonals. The pressure on the area enclosed by the rectangle *c f h g* acts upwards and tends to break the block



(a)



(b) (c)

FIG. 7

about the line of fracture $x x$, the leverage of the pressure being the distance between the centers of gravity of the rectangles *e f h g* and *a i j c*, or the distance $y y_1$. The point y is the center of gravity of the rectangle *e f h g*.

The greatest bending moment, however, may not be about the axis $x x$, Fig. 7 (*a*), but may exist about the axis $x_1 x_1$, Fig. 7 (*b*). Here the one-half area of the bearing of the load is the triangle *a b c* while the area on which the upward pressure acts is the triangle *d e f*. The location of the line of fracture $x_1 x_1$ is determined by the position of the center of gravity of the triangle *a b c*, and the centers of gravity of both the triangle *a b c* and the triangle *d e f* are determined by drawing from the points *a*, *b* and *d*, *e*, lines to the centers of the sides *b c*, *a c* and *e f*, *d f*, respectively. The lever arm with which the pressure on the area *d e f* acts is the distance between the centers of the triangles, or the distance $y y_1$.

Again, failure may not be along either of the axes $x x$ or $x_1 x_1$, but may exist along the axis $x_2 x_2$, Fig. 7 (*c*), in which case the area of pressure from beneath is the rectangular surface *e f g h*, and the moment of this pressure about the line $x_2 x_2$ is equal to its amount multiplied by the lever arm $y y_1$.

While these three cases are assumed in order to determine around which axis the greatest bending moment occurs, it must be understood that the reaction or pressure from the wall beneath acts over the entire bearing bed of the stone.

17. When the maximum bending moment has been determined, such a thickness for the bearing stone must be chosen that the resisting moment will equal the bending moment. The method of determining the resisting moment of rectangular beams is described in *Beams and Girders*, and will be applied in the following examples and illustrations of the method of proportioning bearing blocks.

EXAMPLE.—Fig. 8 (*a*) shows a plan view and (*b*) an elevation of a stone templet, 12 × 24 × 8 inches, supporting the end of an I beam

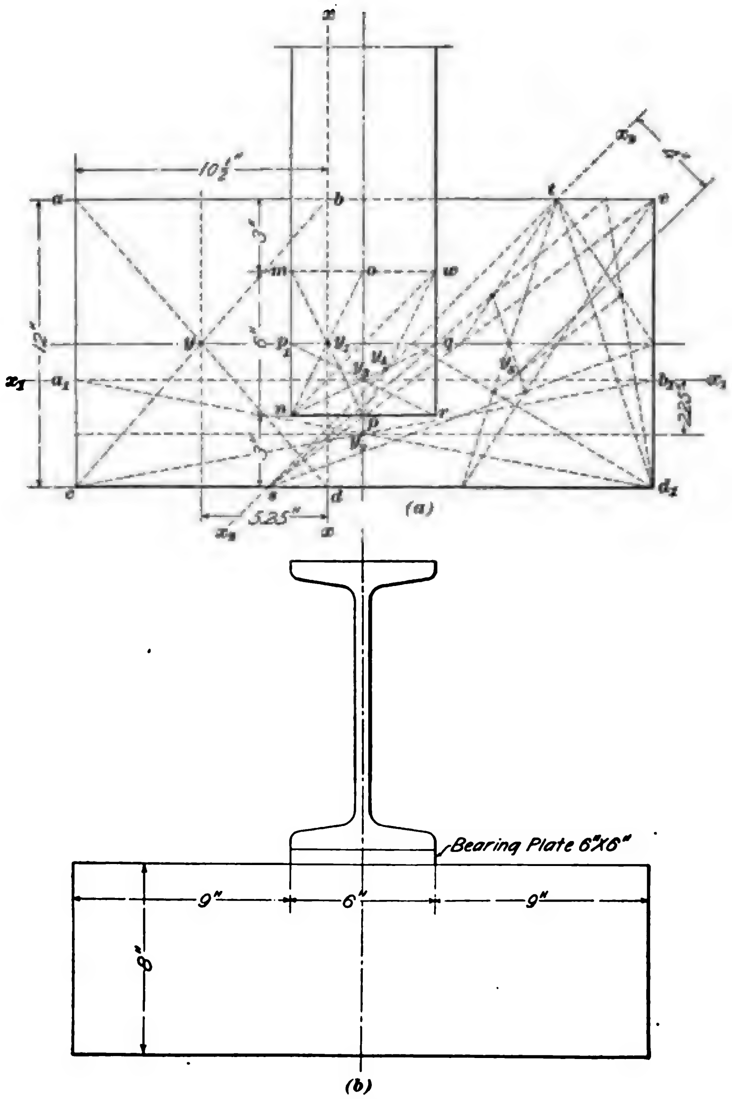


FIG. 8

at which the reaction is equal to 35,000 pounds. This beam is supported on a bearing plate 6 inches by 6 inches, located centrally on the stone. (*a*) What will be the greatest bending moment of the block? (*b*) Will the bluestone templet be sufficiently strong to resist the bending moment?

SOLUTION.—(*a*) It is necessary to analyze the stone for bending about the three axes $x\,x$, $x_1\,x_1$, and $x_2\,x_2$ to determine the maximum. The bending moment about the axis $x\,x$ is equal to the pressure on the area $a\,b\,d\,c$ multiplied by the distance from y to y_1, which is the leverage of the pressure from beneath on this area; this lever arm is equal to the distance between the centers of gravity of the rectangles $a\,b\,d\,c$ and $m\,o\,p\,n$, or by measurement, 5.25 in. The unit pressure on the bed of the block is equal to the load divided by the area of the bed of the block, viz.: $\frac{35,000}{12 \times 24} = 121.5$ lb. Since the area of the rectangle $a\,b\,d\,c$ is equal to 126 sq. in., the upward pressure tending to break the stone about the axis $x\,x$ is equal to $121.5 \times 126 = 15,309$ lb., and the moment of the force is equal to $15,309 \times 5.25 = 80,372$ in.-lb.

To find the bending moment about the axis $x_1\,x_1$, find the area of the rectangle $a_1\,b_1\,d_1\,c$ and the length of the lever arm $y_2\,y_3$, which is the distance between the centers of gravity of the rectangles $a_1\,b_1\,d_1\,c$ and $n\,p_1\,q\,r$. The area subjected to pressure tending to fracture the stone on the axis $x_1\,x_1$ is equal to 108 sq. in. The total pressure on this surface is equal to $108 \times 121.5 = 13,122$ lb. The moment of this force about the axis $x_1\,x_1$ is equal to $13,122 \times 2.25 = 29,524$ in.-lb.

The final moment to determine is the bending moment about the axis $x_2\,x_2$. The upward pressure tending to break the stone about this axis is equal to the unit pressure of 121.5 lb. multiplied by the area of the irregular polygon $s\,t\,v\,d_1$. The area on which the pressure acts is 120 sq. in., so that the upward pressure about the axis $x_2\,x_2$ is equal to $120 \times 121.5 = 14,580$ lb. The center of gravity of the irregular polygon is determined by dividing the figure into two systems of triangles and finding the centers of gravity of the triangles, the intersections of lines connecting the centers of gravity of the two systems of triangles giving the center of gravity of the figure, as described in *Properties of Sections*. In this case, the distance between the center of gravity y_4 of the triangle $n\,w\,r$ and the center of gravity y_5 of $s\,t\,v\,d_1$ is equal to 4 in., so that the bending moment about the axis $x_2\,x_2$ is equal to $14,580 \times 4 = 58,320$ in.-lb.

Therefore, the bending moments about the several axes $x\,x$, $x_1\,x_1$, and $x_2\,x_2$ have been found to be as follows: Moment about axis $x\,x = 80,372$ in.-lb.; moment about axis $x_1\,x_1 = 29,524$ in.-lb.; moment about axis $x_2\,x_2 = 58,320$ in.-lb.; from which it is observed that the greatest bending moment exists about the axis $x\,x$, and is equal to 80,372 in.-lb. Ans.

(*b*) Considering the resistance of the bearing block, it is assumed that the ultimate modulus of rupture of the material is 2,700 lb. per sq. in. and that, allowing a factor of safety of 8, the safe unit stress is 337 lb. The section of the block along the axis $x\,x$ is 8 in. deep and 12 in. wide, so that the section modulus of the bearing block determined by the formula $S = \frac{b\,d^2}{6}$ is equal to $\frac{12 \times 8 \times 8}{6} = 128$. When S equals 128 and s_a, or the safe unit stress, equals 337, the safe resisting moment of the block equals $337 \times 128 = 43{,}136$ in.-lb. This result shows, when compared with the greatest bending moment of 80,372 in.-lb., that the bearing block has an insufficient thickness so that its depth must be increased. Ans.

To determine the thickness of the stone requisite to resist the bending moment, the formula $d = \sqrt{\frac{6\,M}{s_a\,b}}$ could have been used. In this formula d equals the thickness or depth of the rectangular section, M the bending moment in inch-pounds, s_a the safe unit stress, and b the breadth or width of the section. By substitution in the formula,

$$d = \sqrt{\frac{6 \times 80{,}372}{337 \times 12}} = 11.9, \text{ or practically 12 in. Ans.}$$

18. Block Eccentrically Loaded.—The principles applied in determining the maximum bending moment on a bearing block, on which the load is not centrally located, are the same as those described in Art. **16** for a block sustaining a load situated at the center. But the problem is complicated from the fact that when the center of pressure of the load does not coincide with the center of the block, the pressure on the bearing area of the block is not uniform in its intensity. In Fig. 9 is shown a bearing block 16 inches wide, 28 inches long, and 10 inches deep, supporting a plate girder, the reaction from which at the end is equal to 30,000 pounds; the center of effort of this load is located on the bearing block 12 inches from the left-hand end. The center of gravity of one-half of the bearing plate is contained within the vertical line $x\,x$, which, in the problem, will be considered as the line of fracture, or the axis around which the bending moment will be taken. From Fig. 9, the distances l, l_1, and l_2 are found to equal, respectively, 28, 12, and 16 inches. Since these distances and the load are known, the values of k_1 and k_2 may be calculated by

formulas 1 and 2, when k_1 is found to equal 1,530 pounds and k_2 612 pounds. These two values having been obtained, they may be laid off to scale at *i k* and *m n*, when

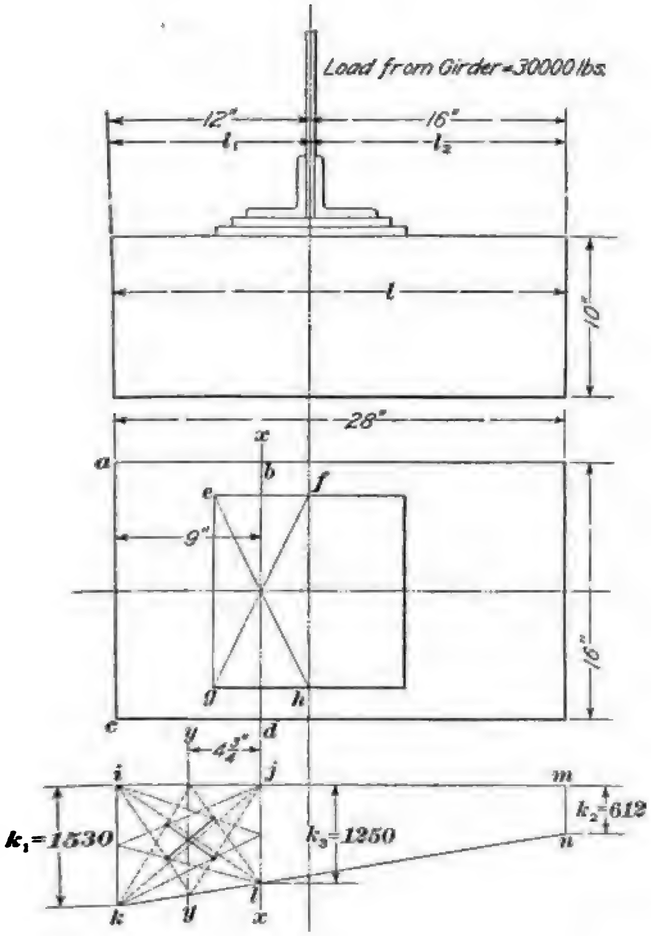


FIG. 9

the figure *i m n k* may be drawn. The ordinate *j l*, which coincides with the axis *x x*, may be measured and the value of k_3 found; in this case it is equal to 1,250 pounds. Since the values of k_1 and k_3 represent the intensity of pressure on

the bed of the bearing block for each lineal inch, the trapezoid $i\,j\,l\,k$, being accurately drawn to scale, represents truly the amount of pressure on the portion of the block included within the rectangle $a\,b\,d\,c$ of the plan. The average pressure per lineal inch on this rectangular surface is equal to $\frac{k_1 + k_2}{2} = 1{,}390$ pounds. This value when divided by the width of the block, or 16 inches, will give a mean unit pressure per square inch on the bed, of approximately 87 pounds. The area of the rectangle $a\,b\,d\,c$ is equal to $9 \times 16 = 144$ square inches, and the total pressure on this area amounts to $144 \times 87 = 12{,}528$ pounds. The center of effort of this load is located at the center of gravity of the trapezoid $i\,j\,l\,k$. A line $y\,y$ passing through the center of gravity of this figure is located $4\frac{3}{4}$ inches from the line $x\,x$. The bending moment about the line $x\,x$ will, therefore, be equal to the total pressure on the area $a\,b\,d\,c$ multiplied by its lever arm, or $12{,}528 \times 4.75$, which gives 59,508 inch-pounds. Consequently, the bearing block must have a resisting moment about the axis $x\,x$ equal to that amount.

19. The maximum bending moment may not exist about the axis $x\,x$, Fig. 9, but may be created about an oblique axis $x_1\,x_1$, Fig. 10. The first difficulty encountered in determining the bending moment about such an axis is finding the entire pressure on the polygon $o\,p\,q\,r$. Before finding this pressure, the axis $x_1\,x_1$ must be located. The position of this axis is determined by drawing a line through the center of gravity of one-half the area of the bearing plate, which is designated in the plan by $a\,c\,e$, the axis being drawn parallel with the diagonal $a\,e$. Not only must the pressure on the area $o\,p\,q\,r$ be found, but the leverage of this pressure, or its perpendicular distance from the center of action to the axis $x_1\,x_1$ must be determined. To find the perpendicular distance between the axis $x_1\,x_1$ and the line $y\,y$ passing through the center of effort of the pressure on the polygon $o\,p\,q\,r$ and parallel to the line $x_1\,x_1$, the figures $i\,f\,g\,h$ and $k\,l\,m\,n$ must be drawn. The trapezoid $i\,f\,g\,h$ is described

in the same manner as the trapezoid *i k n m*, Fig. 9, by laying off k_1 and k_2, in this view represented by the lines *i f* and *h g*, respectively, which must be determined by formulas **1** and **2**; the distance between the ordinates *i f* and *h g* is equal to the length of the block. The block is divided into vertical strips, 1 inch wide, numbered *1, 2 . . . 20* and the dividing lines are carried down to the trapezoid *i f g h*.

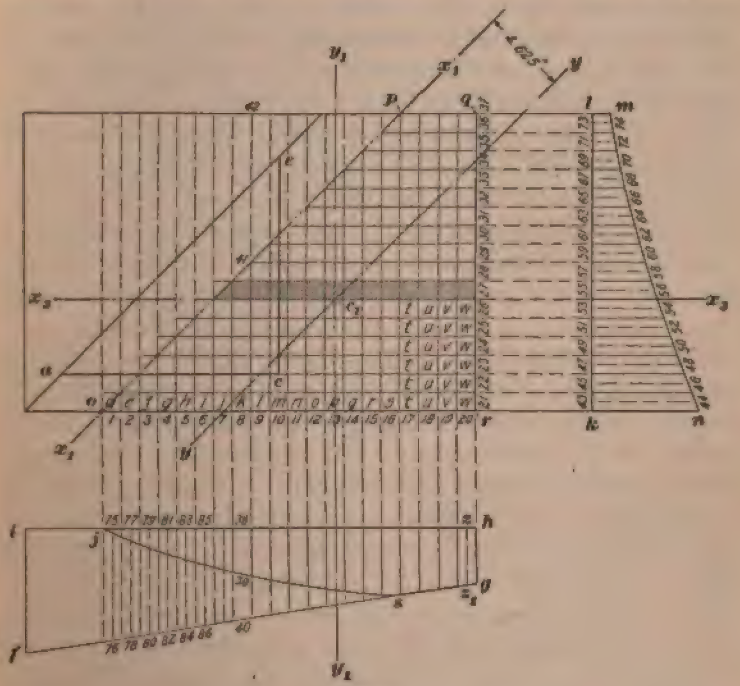


FIG. 10

The portion of each line enclosed by the latter represents the pressure per lineal inch along that particular line. To obtain the *average* pressure of any of these strips, as, for instance, strip *1*, an ordinate *75–76* must be drawn half way between the adjoining ordinates, representing the border lines of the strip. These ordinates *75–76*, *77–78*, etc. would represent the true pressure per lineal inch in any part of the block, provided the whole width *r q* were taken into consideration.

But the area *opqr* is not uniform in width along vertical lines except for the distance *pq*, for, from point *p* to the point *o* the width of the area along the vertical ordinates varies from the width of the block, or 16 inches at *p*, to 0 at *o*. The trapezoid *ifgh* represents the lineal pressure on the bearing block, so that in order to represent diagrammatically the variation in pressure on the area *opqr* in a direction parallel with the length of the block, a line *js* must be described, the figure *jhgs* being thus obtained. By drawing the curved line *js*, the variation in area, as well as in pressure, is diagrammatically shown. In drawing the trapezoid *ifgh*, the area of the bearing surface was not considered because the ordinates in this trapezoid represent the pressure per lineal inch of length, the width being uniform throughout the block. Referring to this trapezoid in connection with the plan of the bearing area *opqr*, it may be readily assumed that the portion of the original trapezoid included in the length *sg* represents the pressure on the portion of the area included in the length *pq*. It can also be assumed that the point *j* is at the intersection of a vertical line drawn from *o*, with the line *ih*, from the fact that this is the termination of the area *opqr*. The positions of the points intermediate between *j* and *s*, such as *39*, may be found by making the ratio between the ordinates *39–40* and *39–38* equal to the ratio of the ordinates *41–42* and *9–41* of the plan. After several of these points have been ascertained they may be connected by the curved line *js*; then those parts of the ordinates situated above this curve will represent the pressure per lineal inch of the corresponding strips in the plan. The center of gravity of the figure *jhgs* may be found by any of the several methods for determining the centers of gravity of irregular figures, and the line $y_1 y_1$ thus obtained. This line $y_1 y_1$ will contain in its length in the plan, the center of effort of the pressure on the area *opqr*. The exact position of this point, however, will have to be obtained from the diagram of pressure, *klmn*, made transversely, or across the width of the block. This diagram is not so readily drawn as the one at *jhgs*, for the pressure

on each lineal inch of width must first be determined. The ordinates *22*, *23*, *24*, *25*, etc. are first drawn, spaced at a unit measurement apart, then the area *o p q r* will have been divided into a number of unit areas, which, in the problem in hand, have been made equal to 1 square inch. By scaling each of the ordinates in the diagram *i f g h* and dividing by the width of the block, or in this case 16, the value of each square, as *t*, *u*, *v*, *w*, is obtained. For instance, if it is desired to ascertain the pressure per square inch of the strip *20*, the total pressure on the strip is first found by measuring the length of the ordinate $z z_1$ and multiplying this length by the number of pounds assumed per unit length. Dividing this pressure by 16 gives the pressure of each square *w*. By adding the values of all the units of area *w*, *v*, *u*, *t*, etc. across each strip, one of which is shown shaded, the amounts for the ordinates *43–44*, *45–46*, *47–48*, etc. are determined.

20. In Table I, which gives the necessary calculation for these ordinates by which the points *44*, *46*, *48*, etc. are found and the curve *n m* drawn, the values in column (*a*) are obtained by scaling the mean ordinates *75–76*, *77–78*, *79–80*, etc. of the diagram for the pressure longitudinally. In column (*b*) is given the width of the block and in column (*c*) are the results obtained by dividing the pressure per lineal inch of length by the width of the block, which gives the pressure on the unit areas *w*, *v*, *u*, *t*, etc. In column (*d*) are given the reference letters by which the various squares are marked in the figure *o p q r*. In column (*e*) are given the sums of the values of the unit areas of each strip extending longitudinally across the block, including the pressure on the entire unit cut by the axis x_1 x_1; that is, the value 735.39 is the value of the shaded strip shown in the plan *o p q r*, plus one-half a unit area, and represents, accurately enough for all practical purposes, the total pressure on the area included in this strip. It will be noted by referring to the diagram that the shaded portion includes thirteen full units and a half unit, and in each

case the value of the half unit area formed by the oblique axis $x_1 x_1$ must be added in order to obtain the full value of each strip. In column (*f*) are given the values of one-half of the unit area cut by the axis $x_1 x_1$, these

TABLE I

Values Obtained by Scaling Mean Ordinates of Diagram for the Longitudinal Pressure (*a*)	Width of the Block (*b*)	Pressure On the Unit Areas (*c*)	Reference Letters of Square (*d*)	Sum of the Unit Areas in a Longitudinal Strip (*e*)	One-Half of the Unit Area Cut by $x_1 x_1$ (*f*)	Result Obtained by Subtracting Values in (*f*) From the Values in (*e*) (*g*)
1,200.00	16	75.00	*d*	1,158.39	37.50	1,120.89
1,171.25	16	73.20	*e*	1,083.39	36.60	1,046.79
1,142.50	16	71.40	*f*	1,010.19	35.70	974.49
1,113.75	16	69.60	*g*	938.79	34.80	903.99
1,085.00	16	67.80	*h*	869.19	33.90	835.29
1,056.25	16	66.00	*i*	801.39	33.00	768.39
1,027.50	16	64.21	*j*	735.39	32.10	703.29
998.75	16	62.41	*k*	671.18	31.20	639.98
970.00	16	60.60	*l*	608.77	30.30	578.47
941.25	16	58.82	*m*	548.17	29.41	518.76
912.50	16	57.00	*n*	489.35	28.50	460.85
883.75	16	55.23	*o*	432.35	27.61	404.74

found and the line $x_1 x_1$ drawn through this point. The intersection of the axis $x_1 x_1$ with $y_1 y_1$ at c_1 will give the center of effort of the pressure on the area *o p q r*. The perpendicular distance between the line *y y* and the axis $x_1 x_1$ is the lever arm of the pressure on the portion of the bearing area *o p q r* and the amount of this pressure multiplied by the lever arm will give the desired bending moment about the axis $x_1 x_1$. The area of the figure *k l m n* should equal the area of the figure *j h g s*, for both of these diagrams represent accurately the entire pressure on the area *o p q r*. The total pressure on this area is found to equal 10,049.55 pounds, so that the bending moment will equal 10,049.55 × 4.625, or 46,479 inch-pounds. This is less than the bending moment about the axis *x x*, Fig. 9, so that evidently the greatest bending moment was found in the calculation in Art. **18**.

The method just described is somewhat complicated and need only be applied under the unusual conditions when a principal girder of great structural importance is, for unavoidable reasons, placed eccentrically on the bearing block and where the possible thickness of the bearing block requires close figuring.

BEARING BLOCKS SUBJECTED TO INCLINED LOADS

21. The structural feature of a bearing block supporting an inclined load is not very common, but problems occur occasionally that require the application of the principles involved in this discussion. The analysis of such a problem is best explained by assuming some such practical example as the following, and working through the several steps to the solution:

EXAMPLE.—A limestone bearing block 16 in. × 16 in. × 9 in. thick supported on a brick wall laid up in cement mortar supports at the center a load of 4,000 pounds. The load is inclined in a plane parallel with one of the sides at an angle of 25° with the vertical. (*a*) Will the block in question be secured against sliding? (*b*) Are its bearing area and resisting moment sufficient?

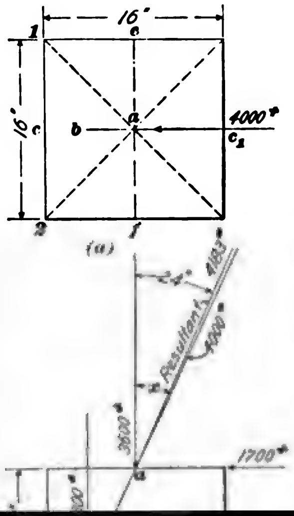
SOLUTION.—(*a*) The conditions of the problem are shown in Fig. 11, in which (*a*) is a plan view and (*b*) an elevation. The first step in the

solution is to find the horizontal and vertical components of the inclined load or oblique thrust.

Trigonometrically, the vertical component equals the product of the load and the cosine of the angle x, while the horizontal component equals the product of the load and the sine of the angle x. Thus, vertical component = .90631 × 4,000 = 3,625.24 lb.; horizontal component = .42262 × 4,000 = 1,690.48 lb.

For convenience in the solution of the problem, these values may be taken in round numbers at 3,600 and 1,700 lb., respectively.

The tendency to slide or shear the stone on its bed is caused by the horizontal component of the load, and in order to determine whether the stone is stable against this tendency, the severest condition that may exist, namely, that the block is bedded in wet mortar, will be considered. The shear of 1,700 lb. at the top of the stone will be transmitted to the bed joint of the bearing block by the cohesion of the mortar or by friction. The resistance of the bearing block to sliding is equal to the vertical pressure on the bed multiplied by the coefficient of friction of the material. The coefficient of friction for masonry laid in wet mortar is equal to .47, while the total vertical pressure on the bearing area of the block is equal to the sum of the vertical component of the oblique force and the weight of the block, or 3,800 lb.



By calculation then the frictional resistance to sliding is 3,800 × .47 = 1,786 lb., which is in excess of the sliding or shearing tendency of 1,700 lb. The factor

[redacted]

may be determined graphically in the usual manner. The point b is at the intersection of the bed joint and this resultant.

The center of application, or the center of pressure, not being coincident with the center a of the center line $c c_1$, the pressure will not be uniform in this direction, but will vary between points c and c_1. A uniform pressure will exist, however, along the line $e f$, or along any line parallel with it.

Referring to Fig. 12, it will be observed that the point c is assumed as $\frac{1}{2}$ in. inside the edge of the block, from the fact that the efficient bearing area does not approach nearer to the edge of the block than this. The distance between the point b and the vertical line through a may be found graphically by drawing the stone to scale and extending the resultant until it intersects the base line. Or, it may be found trigonometrically. In either case, it is found that b will be located 4 in. to the left of the vertical center line; $c b$ is therefore equal to $8 - 4 - \frac{1}{2} = 3\frac{1}{2}$ inches, which distance is less than one-third the length of the block. This condition existing, and tension in the joint being neglected, the pressure on the block may be represented by a triangle, as explained in connection with Fig. 1 (d). To lay out this triangle $c d e$, Fig. 12, the ordinates of which represent the pressure on the bearing block per unit of lineal measurement, the maximum pressure per inch at the point c must be obtained by formula **3** and the amount laid off to scale from c to e, this distance being represented by k_1.

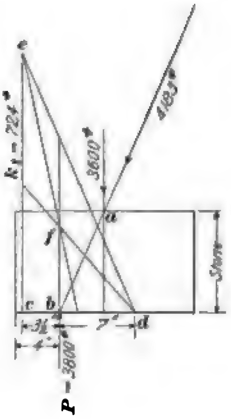


FIG. 12

By substitution in formula **3**, $k_1 = \frac{2 \times 3{,}800}{3 \times 3.5} = 723.80$ lb., or, approximately, 724 lb. This amount is the pressure per unit of lineal measurement at the point c; the maximum unit pressure p on the bearing block is equal to the maximum pressure per unit of length divided by the width of the block.

The efficient width of the block equals 15 in.; so that

$$P = \frac{724}{15} = 48.27 \text{ lb.}$$

As the center of resistance is considered to be in line with the center of gravity of the triangle, the distance from the edge c to the point of application b is one-third the base of the triangle and, consequently, the remaining two-thirds of the base $c d$ equals $2 \times 3\frac{1}{2} = 7$ inches. The triangle may then be completed by drawing the line $e d$.

Taking 1,500 as the average ultimate strength per square inch of the masonry, the factor of safety with regard to the bearing of the block is

equal to $1{,}500 \div 48.27 = 31$, which shows the masonry to have a safe crushing strength greatly in excess of the maximum unit pressure. Ans.

In Figs. 11 and 12, the force of 4,000 lb. is considered as applied at a single point. The theoretical bending moment will then be the moment of the total pressure on the joint of the bearing block between c and d, Fig. 12, with a lever arm equal in length to the distance between the center of application of the pressure at a and a line passing through the center of gravity of the triangle $c\,e\,d$. This triangle represents diagrammatically the amount of the pressure or reaction on the joint between the points c and d, or 3,800 lb.; and its center f is found by drawing lines from two of its apexes to the middle of the opposite sides. The perpendicular distance from the center of pressure a to a vertical line passing through the center f of the triangle and also through the point b, has been found to be 4 inches. Consequently, the bending moment, or M, about $a = 3{,}800 \times 4 = 15{,}200$ in.-lb. If a factor of safety of 10 is desired and the ultimate transverse strength of limestone is taken at 1,500 lb. per sq. in., the safe unit stress will amount to 150 lb.; and as the safe resisting moment equals the section modulus multiplied by the safe unit stress, by transposing, the required section modulus will be equal to $15{,}200 \div 150 = 101$. The section of the bearing block along the line of fracture passing through the point a and parallel with the edge of the block marked *1–2* in the plan, Fig. 11, has a section modulus equal to $\frac{16 \times 9 \times 9}{6} = 216$, which shows a great excess of transverse strength over the bending moment. Ans.

EXAMPLES FOR PRACTICE

1. A stone templet supports one end of a plate girder carrying a uniformly distributed load of 60,000 pounds. If the stone is to be 14 inches wide, what must be its length, providing the masonry on which it is sustained will safely carry 100 pounds per square inch?

Ans. 21.43 in.

2. The capstone on a pilaster sustains a load of 20,000 pounds. The center of action of the load lies in the center line that is at right angles to the wall, at a distance from the front edge equal to 8 inches. The width of the stone in a direction parallel with the wall is 18 inches, and the depth of the block is 20 inches. What will be the maximum unit pressure on the bed of the stone? Ans. 88.88 lb.

3. In Fig. 13 is shown the diagram of a stone templet that supports one end of a plate girder, the abutment reaction of which is 30,000 pounds. What are the bending moments about the axes xx, x_1x_1, and x_2x_2?

$$\text{Ans.}\begin{cases}54{,}100 \text{ in.-lb.}\\22{,}500 \text{ in.-lb.}\\36{,}500 \text{ in.-lb.}\end{cases}$$

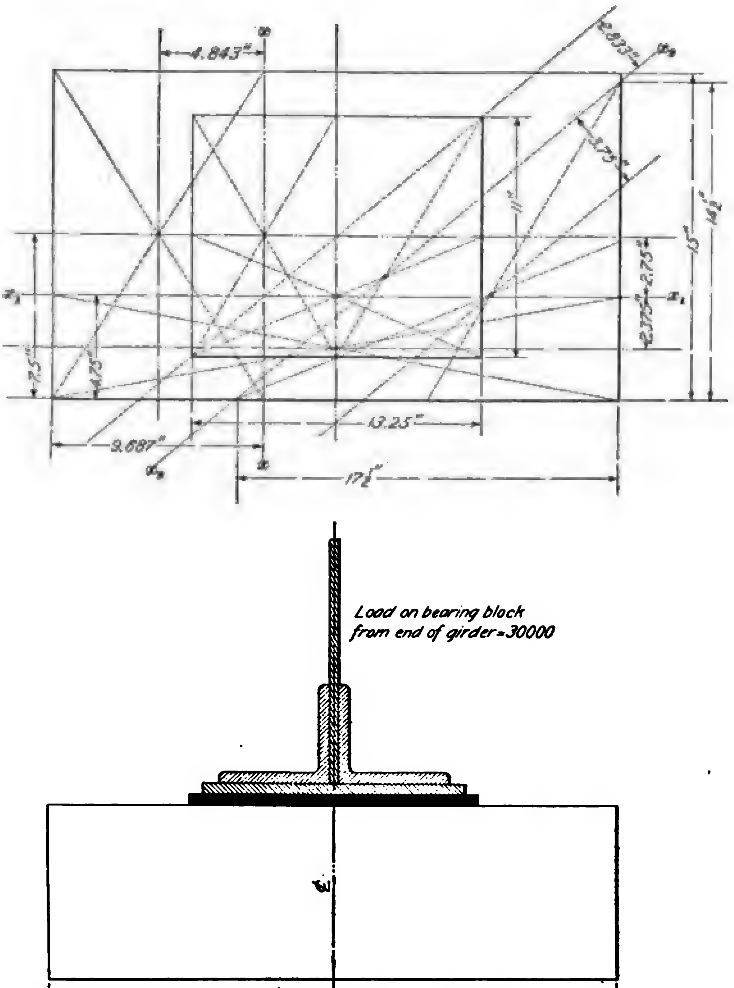

4.843"
2.853"
3.75"
11"
15"
14½"
2.75"
2.375"
7.5"
4.75"
13.25"
9.687"
17½"
Load on bearing block
from end of girder = 30000
26"

FIG. 13

COLUMNS

22. Monolithic columns are seldom used in modern work for any other purpose than architectural embellishment. In colonnades, the columns only support the entablatures and the weight of some portion of a roof, if such exists; the combined weight on the column is usually a very small proportion of its ultimate strength. If a monolith, or single stone, is used as a mullion, the weight of the wall is generally supported by relieving arches or steel lintel beams, so that the mullion sustains only its own weight and possibly the weight of a face lintel, which is purely an architectural feature without structural purpose.

Where a monolithic column is a structural member supporting a considerable load, and its height does not exceed 10 times its least dimension or diameter, the strength of the column may be calculated by considering the ultimate direct compressive strength of the material. In proportioning such columns, the factor of safety should not be less than 10, and preferably should be 15.

In the absence of reliable data from experimental tests on full-size specimens, as have been obtained for steel and wooden columns, by analysis a monolithic column becomes the same as a bearing block, the column being, however, subjected to only a uniform direct compressive stress.

The required section at the base of the column, in square inches, is obtained the same as the bearing area of a block, by dividing the load, in pounds, by the safe unit crushing strength of the mortar or cement used in setting the column. For this reason, monoliths supporting great loads should be set in neat Portland cement with fine joints. The stones should, where possible, be set on a natural bed, that is, with the grain horizontal, to develop their greatest strength as well as to resist the action of the weather and the disintegrating effect of time, though it is usual to cut stone columns with their grain, or cleavage, parallel with their longest axis; stones of this character should never be used as columns without allowing an extra large factor of safety.

Care should be taken that the bearing surfaces of stone columns lie in horizontal planes at right angles to the axis of the column, for only by this means will the loads be uniformly distributed over their bearing areas. If the bearing is cut concave, as shown in Fig. 14, the mortar will compress and give the greatest bearing on the edge, which will tend to spall the stone at the joint. Should the bearing be cut convex, the greatest pressure would be at the center, which under a heavy load would tend to crack the base or bearing stone. It is common practice with stone cutters to cut the joint concave in order to insure a neat joint and easy setting; for this reason the bearing ends of all stone columns should be carefully inspected. The joint at the base and cap of monolithic columns should not be less than $\frac{3}{16}$ inch thick.

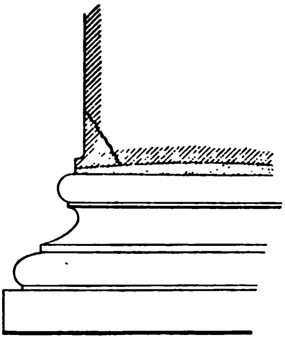


FIG. 14

LINTELS

23. Lintels are stone beams spanning a clear space, such as a window opening or doorway. They are used

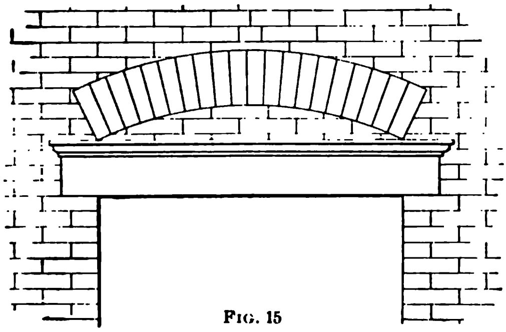


FIG. 15

primarily for architectural reasons; that is, because they are requisites of the style of architecture employed. From a constructive point of view, arches should be used in preference to lintels, if the abutments are sufficient to withstand

the horizontal thrust, from the fact that arches possess greater strength and may usually be had for openings of large span at a less cost. Where there is a considerable load over an opening in a wall and a flat soffit is required, instead of proportioning a stone beam to sustain the transverse stress, it is better to use a relieving arch, as shown in Fig. 15, and require the stone beam to support only that portion of the wall filling in the **tympanum,** or the space included between the soffit of the arch and the upper surface of the lintel.

CORBELING ACTION OF SUPERIMPOSED MASONRY

24. In Fig. 16 is shown a stone lintel over a door opening. The opening is without a frame, and the lintel is the full thickness of the wall in width. It is assumed that there are no openings through the wall directly over the lintel, so that even if the lintel were removed, the brickwork would arch or **corbel** itself above the opening, owing to the fact that each course is set in mortar and bonded. The bricks in each course will project slightly beyond the course below and the line of fracture that will exist in ordinarily good masonry, if the lintel is removed after the mortar has set, will correspond with the lines $a\,b$ and $b\,c$.

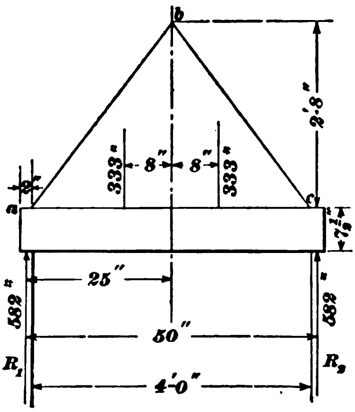


Fig. 16

The angle of the corbeling, and, consequently, the amount of the wall to be carried by the lintel, depends on the character of the material and workmanship, but it is usually safe practice to allow two-thirds the span as the altitude of the triangle $a\,b\,c$ for brickwork, though it is not uncommon, where the workmanship and materials are of the best quality, to allow as low as one-third the span for the height of the triangle.

Stone walls corbel over openings in the same way, the angle of the corbeling depending on the shape of the stones and the manner in which they are laid. In stonework, it would usually be considered safe practice to assume this same proportion of height to span for the triangular weight on the lintel.

25. In analyzing the constructive features of a lintel, two points must be determined: first, the consideration of the bearing of the lintel at its ends, and second, whether the lintel has sufficient transverse strength to resist the bending action produced by the load.

In the problem imposed by the conditions shown in Fig. 16, let it be assumed that the wall supported by the lintel is of brick 13 inches in thickness and that the brickwork composing the wall weighs 125 pounds for each superficial foot. The base of the triangular portion of the brickwork supported by the lintel is 4 feet, and if the height or altitude of the triangle is made equal to two-thirds of the base, this dimension will equal 2 feet 8 inches. The area of the triangular portion will then be equal to $\frac{4 \times 2.66}{2} = 5.33$ square feet. For each foot of area the brickwork weighs 125 pounds; in consequence, the total weight of the triangular portion of the wall is $5.33 \times 125 = 666$ pounds. To this the weight of the lintel should be added; and as it contains 2.93 cubic feet, and the weight of the stone is assumed at 170 pounds per cubic foot, the weight of the lintel will equal $2.93 \times 170 = 498$ pounds. The combined weight of the lintel and the portion of the wall supported by it will then equal $498 + 666 = 1{,}164$ pounds. The load is equally distributed each side of the center line; consequently, the reactions R_1, R_2 will each equal one-half of the total load, or 582 pounds. The lintel has a bearing on the wall of 2 inches at each end, and as its width is 13 inches, the total bearing area at the end of the lintel equals 26 square inches. The unit pressure on the brickwork from the ends of the lintel will then equal one-half the total load divided by the

bearing area at one end, or $\frac{582}{26}$, which gives 22.38 pounds. This value is well within the safe bearing value of the most ordinary brickwork, so that the bearing of the lintel on the wall is sufficient.

The greatest bending moment on the lintel occurs at the center of the span and is equal to the algebraic sum of the moments of the loads and reactions about this point. The conditions of the load when the weight of the lintel is taken into account are best shown in Fig. 16. Here the positive moment about the center of the span is equal to the amount of the reaction R_1 multiplied by the distance from its line of action to the center of moments, or 25 inches; while the negative moment about the same point is equal to the weight of one-half of the lintel by the distance from its center of gravity to the center of the span, plus the moment of one-half the triangular portion of the wall supported on the lintel, by its lever arm. The positive moment then equals $582 \times 25 = 14{,}550$ inch-pounds. The negative moments are observed on referring to Fig. 16 to be as follows:

Portion of lintel equals 249×13	$= 3237$	inch-pounds
Portion of load equals 333×8	$= 2664$	inch-pounds
Total negative moments	$= 5901$	inch-pounds

The algebraic sum of the positive and negative moments equals $14{,}550 - 5{,}901 = 8{,}649$ inch-pounds.

26. To determine whether the lintel is sufficiently strong to sustain this transverse stress, it is first necessary to figure the section modulus by the formula

$$S = \frac{b d^2}{6}$$

in which S = section modulus;
d = depth of beam in inches;
b = breadth or width of beam in inches.

The width of the lintel is 13 inches and its depth 7.5 inches, so that the section modulus, or

$$S = \frac{13 \times 7.5 \times 7.5}{6} = 121.87$$

The safe resistance of the lintel may be found by the formula

$$M_1 = S s_a,$$

in which M_1 = resisting moment;

s_a = safe unit stress.

When it is assumed that the lintel is to be of Vermont marble, its ultimate modulus of rupture is 1,200 pounds; hence, if a factor of safety of 10 is employed, its safe unit stress, or s_a, will equal 150 pounds. By substitution, therefore, $M_1 = 121.87 \times 150 = 18{,}280$ inch-pounds. A comparison of this value just found with the transverse stress on the stone shows that the lintel is amply strong.

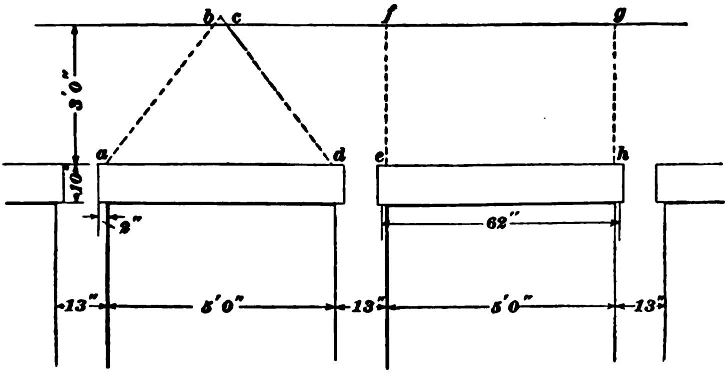


FIG. 17

EXAMPLE.—Stone lintels are used over a series of openings in a 9-inch wall, as designated in Fig. 17. The wall supported on the lintels is a parapet wall and extends 3 feet above the top of the lintels. Providing the weight of the 9-inch wall per superficial foot is 90 pounds, will a bluestone lintel 10 inches deep be sufficiently strong to resist the transverse stress?

SOLUTION.—The wall will probably corbel along the lines ab and cd, but it is advisable, from the fact that the wall is not sufficiently high to allow the apex of the triangle to fall entirely within its surface, to figure the weight on the lintel included within the rectangle $efgh$. The width of the rectangular surface is equal to the span of the opening, or 5 ft., and its height, as shown in the figure, is 3 ft., so that its area is equal to 15 sq. ft. Since the weight of the brickwork has been assumed as 90 lb. per superficial foot, the load on the lintel will equal $15 \times 90 = 1{,}350$ lb. The depth of the lintel has been

assumed to equal 10 in., and as its width is 9 in. and its length 64 in., its cubical contents will equal $\frac{10 \times 9 \times 64}{1,728} = 3.33$ cu. ft. Bluestone weighs about 160 lb. per cu. ft., so that the weight of the lintel will equal $3.33 \times 160 = 533$ lb. The total load supported by the transverse strength of the lintel is, in consequence, equal to the weight of the brickwork, or 1,350 plus the weight of the lintel, or 533, which gives 1,883 lb. The load on the lintel may be considered as uniformly distributed and the bending moment, or M, may be found by the formula

$$M = \frac{W l}{8}$$

in which W = total load;

l = length in inches.

By substitution, $M = \frac{1,883 \times 62}{8} = 14,593$ in.-lb. It will be noticed that the length of the span is taken between the centers of bearings.

The relation between the bending moment and the resisting moment is expressed by the formula $M = M_1$, while the value of the resisting moment M_1 for a rectangular section may be obtained from the equation $M_1 = S s$, or

$$M_1 = \frac{s b d^2}{6}$$

in which s = modulus of rupture;

b = breadth or width of beam in inches;

d = depth of beam in inches.

The modulus of rupture for bluestone, considering good average values, may be taken at 2,700 lb., so that if a factor of safety of 10 is used, the safe unit stress, or $s_a = 270$ lb. By using this value in the above formula instead of s, the ultimate unit stress, the safe resistance of the beam will be obtained. By substitution, $M_1 = \frac{270 \times 9 \times 10 \times 10}{6} = 40,500$ in.-lb. A comparison of this value, which represents the safe resistance of this stone, with the bending moment, or 14,593, shows that the bluestone lintel only sustains about one-third of the load which it is capable of supporting with safety, and that its depth could be reduced if the architectural effect were not to be considered. Ans.

TRANSVERSE RESISTANCE OF THE SUPERIMPOSED MASONRY

27. Though it is good practice to consider the load on the lintel as equal to the weight of a triangular piece of masonry having a base equal to the span of the lintel and a height of two-thirds of the base, it is not, however, always

economical, and its fallacy is demonstrated when the observation of old work reveals the fact that the masonry above openings is retained in position without extraneous support. Masonry that has properly set has considerable transverse strength, so that if the height of the wall above the lintel is sufficient there will be no load on the lintels and it will be useful only as a finish and as a support for the few bricks or stones that might loosen from the under side of the masonry.

The height above a lintel of any wall that will have sufficient transverse strength to support its own load may be determined from the general formula expressing the relation between the bending moment and the moment of resistance of a rectangular section, or $M = M_1$. If the lintel is securely built in at the ends it may be considered as a beam fixed at both ends and supporting a uniformly distributed load.

The formula for the bending moment on a beam of this character and under this condition of loading is

$$M = \frac{WL}{12}$$

in which M = bending moment;
L = span of the opening, in feet;
W = weight of load.

The weight W is determined by multiplying the volume of the wall by the unit weight, or, expressed as a formula,

$$W = (TLH)w \qquad (5)$$

in which T = thickness of the wall, in feet;
L = span of the opening, in feet;
H = height of the wall above the lintel, in feet;
w = weight of masonry, in pounds per cubic foot.

By substitution of these values in the formula expressing the bending moment, $M = \frac{(TLH)wL}{12}$, or

$$M = \frac{(TL^2H)w}{12} \qquad (6)$$

The section of the wall subject to transverse stress is rectangular, and as the section modulus for a rectangular section is expressed by the general formula $\frac{bd^2}{6}$, the section

modulus for the wall section, in feet, may be found by the expression $\frac{TH^2}{6}$.

In formula **5**, T, the thickness of the wall, is equal to b, the breadth, and H is equal to the depth d in the general formula. The safe resisting moment is, in consequence, equal to $\frac{144\,s_a\,TH^2}{6}$, in which s_a is the safe modulus of rupture of the material, and the factor 144 is introduced so that the value, 144 s_a, will represent the modulus of rupture in pounds per square foot of section, this being necessary from the fact that T and H are in feet.

28. The relation between the bending moment due to the weight of the section of the wall above the lintel and its resisting moment may now be expressed by the formula

$$\frac{(TL^2H)\,w}{12} = \frac{144\,s_a\,TH^2}{6}$$

By cancelation, $\frac{L^2 w}{12} = 24\,s_a H$, and $H = \frac{L^2 w}{288\,s_a}$.

The value of w, or the weight per cubic foot of the brickwork, varies, being from 112 to 120 pounds for work laid in lime mortar and 130 for work set in cement mortar. Therefore, since the value, 288, is constant, the above formula may be written

$$H = a\frac{L^2}{s_a} \qquad (7)$$

in which the values of a are as follows:

For brickwork in lime mortar, $a = .416$
For brickwork in cement mortar, $a = .451$

29. The value H, or the height of a brick wall that will supply such transverse resistance as to just support its weight, may be found by formula **7**. For any height greater than H, the wall will have more than sufficient strength against failure by bending to support its own weight, while if the height of the wall is less than that determined by the formula, the wall will not support its own weight, and the additional resistance supplied by a lintel will be required.

Where the wall is of less than sufficient height to support its entire weight, for economical reasons the lintel may be proportioned to support only the portion of the load that cannot be supported by the wall. The proportion of the weight of wall supported by the lintel, when the transverse strength of the brickwork is considered and when the height of the wall is less than the distance H, may be determined from the following formulas.

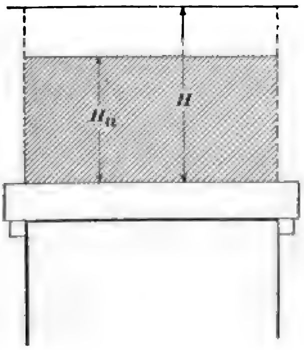


FIG. 18

30. The transverse strength of the wall depends on the section modulus of the wall section, which varies as the square of the height of the wall, or as H^2. It is evident, however, that any change in the height of the wall increases or decreases the weight of the wall proportionately, so that the *actual resistance* of a wall of any assumed height when compared with a wall of any other height varies not as H^2, but as H.

Referring to Fig. 18, let H indicate the height of a brick wall sufficiently great to make the latter self-supporting. It is evident that as the net resistance of the wall varies directly with H, the proportional resisting moment of a wall with the height H_a bears the ratio of $\frac{H_a}{H}$ to the self-supporting wall. This proportion, however, is simply the relation of the transverse strength of the shaded portion, which represents the assumed height of the wall, to the transverse strength of a wall of sufficient height to support its own weight, and it is necessary to find the proportion of the weight of the shaded portion that may be supported by its transverse strength. Therefore, since the weight varies as the height of the wall, or $\frac{H_a}{H}$, H expresses the portion of the wall that is supported by its own resistance, while the height

of the portion of the weight coming on a lintel from a wall of less height than H is found from the formula

$$H_p = \left(1 - \frac{H_a}{H}\right) H_a \qquad (8)$$

31. The weight coming on the lintel will be greatest when the value of H_p is maximum, and this is reached when $H_a = \frac{1}{2} H$. Assume, therefore, that the distance H_a, in Fig. 18, is such that the maximum load on the lintel is realized; then, by substituting in formula **8**, $H_p = \left(1 - \frac{H}{2H}\right)\frac{H}{2}$, or

$$H_p = \frac{H}{4} \qquad (9)$$

From this it is evident that the maximum load on any lintel is equal to the weight of a wall having a height of one-fourth the height of the self-supporting wall, or H.

For convenience, a formula may now be evolved by which the maximum load that can be imposed on any lintel will be found. It has been found that when H_a equals $\frac{H}{2}$ the greatest load exists on the lintel, and from formula **7** it was found that $H = a\frac{L^2}{s_a}$, so that by substitution, where H_a equals $\frac{H}{2}$,

$$H_a = a\frac{L^2}{2s_a} \qquad (10)$$

The maximum weight on the lintel when H_a is equal to $\frac{H}{2}$ is equal to one-half the weight of a wall having a height of H_a, or $\frac{w H_a L T}{2}$, so that if the value of H_a, given in formula **10**, is substituted and W represents the total load, in pounds,

$$W = \frac{w a L^2 T}{4 s_a} \qquad (11)$$

32. From these facts the following data may be deduced:

1. The height of a brick wall that will be self-supporting may be found by dividing the square of the span of the opening, in feet, by the safe modulus of rupture of the

masonry, and by multiplying this result by .416 for brickwork in lime mortar, or .451 for brickwork in cement mortar.

2. The maximum weight on a lintel supporting a brick wall is realized when the height of the wall above the lintel is equal to one-half the height of a self-supporting wall, and this weight is equal to that of a wall whose height is one-fourth the height of a self-supporting wall.

3. The amount of the maximum load, in pounds, that can be imposed on any lintel is equal to the product of the cube of the span, in feet, the thickness of the wall, in feet, and the square of the weight of wall per cubic foot, divided by 1,152 times the safe modulus of rupture of the brickwork.

4. The weight on a lintel from a brick wall not self-supporting is equal to the weight of a wall whose height is the difference between 1 and the fraction determined by dividing the actual height of the wall by the height of wall that would be self-supporting, multiplied by the actual height of the wall.

EXAMPLE 1.—The width of an opening spanned by a lintel is 10 feet. If the brick is laid in cement mortar and has a safe modulus of rupture equal to 10, what should be the height of the wall above the opening in order to relieve the lintel of all the load?

SOLUTION.—By substituting in formula **7**,

$$H = .451 \frac{10 \times 10}{10} = 4.51 \text{ ft.} \quad \text{Ans.}$$

EXAMPLE 2.—A lintel of bluestone supports a brick wall 2 feet thick over an opening 6 feet wide. What will be the maximum load to which the lintel can be subjected if the wall is laid in lime mortar and has a safe modulus of rupture of 7?

SOLUTION.—By substituting in formula **11**,

$$W = \frac{120 \times .416 \times 6 \times 6 \times 6 \times 2}{4 \times 7} = 770 \text{ lb.} \quad \text{Ans.}$$

EXAMPLE 3.—A parapet wall 3 feet high over an opening 10 feet wide is laid up in cement mortar and is supported over the opening by a lintel. Providing the safe modulus of rupture of the masonry is assumed at 5 pounds, what will be the load supported by the lintel, if the wall is 18 inches thick?

SOLUTION.—By substituting in formula **7**, $H = .451 \frac{10 \times 10}{5} = 9$ ft. When H has been obtained formula **8** becomes, by substitution,

$H_P = \left(1 - \frac{3}{9}\right) 3 = 2$ ft. The weight, or load, on the lintel will equal the weight of a brick wall 18 in. thick and 2 ft. high having a length equal to the span, or 10 ft. Consequently, the load supported by the lintel will equal $1.5 \times 2 \times 10 \times 130 = 3{,}900$ lb. Ans.

SUPERIMPOSED WALL WITH WINDOW OPENINGS

33. When the wall supported by the lintel is broken by window openings, as in Fig. 19 (*a*) and (*b*), the assumed load must be decided by the designer. At (*a*), owing to the fact that the wall is broken by the opening, the weight on the lintel will be the weight of the trapezoidal portion contained between the points *a*, *b*, *c*, *d*. In Fig. 19 (*b*), as there are two window openings above the lintel, it is reasonable to figure the load on the lintel as the weight of the irregular shaded portion contained between the points *a*, *b*, *c*, *d*, *e*, *f*, *g*, etc.

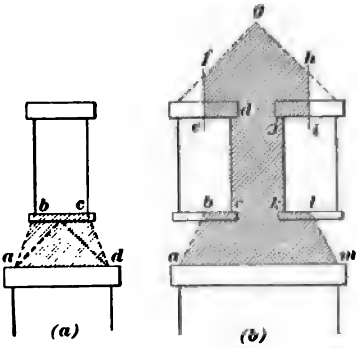


FIG. 19

34. Frequently, the wall, of which the lintel is a structural member, extends at right angles to the direction of the joists or beams and the lintel is required to support their ends. When this condition obtains, the lintel should be proportioned to carry not only the weight of the superimposed wall, but also a portion of the floor load. It is seldom, however, when the lintel is required to support a portion of the floor load, that the structural member is made of stone from the fact that the transverse strength of stone is not great and the heavy load would require a stone out of all architectural proportion and beyond economical consideration.

EXAMPLES FOR PRACTICE

1. A New Hampshire granite lintel weighing 170 pounds per cubic foot, which is the structural support for a 12-inch brick wall laid in cement mortar above an opening 8 feet wide, has an ultimate unit

modulus of rupture of 1,500 pounds. Provided the transverse resistance of the wall is neglected, what factor of safety will a lintel 10 inches in depth develop? Ans. 5.17

2. A brick wall laid in cement mortar is 18 inches thick and is supported over an opening 8 feet wide by a bluestone lintel. What will be the maximum load to be supported by the lintel if the modulus of rupture of the brickwork is taken at 8 and the transverse strength of the wall is considered? Ans. 1,407 lb.

3. A brick parapet wall 3 feet 6 inches high and 18 inches thick laid up in cement mortar is supported by a lintel over an opening 11 feet wide. If the modulus of rupture of the brickwork is taken at 5, what will be the load supported by the lintel? Ans. 5,168 lb.

4. It is desired to cut a 12-foot opening through an old brick wall 21 inches thick, which was constructed with the best Portland cement mortar. The height of the wall from the top of the proposed opening is 15 feet. Considering the transverse strength of the wall as that of a beam merely supported at the ends, what is the unit modulus of rupture to which it is subjected? Ans. 6.5

5. A brick wall 12 inches thick laid in cement mortar is built over a 9-foot opening, the work above the opening being supported on a bluestone lintel having a safe unit strength of 270 pounds. It is assumed that the lintel is not securely built in the wall at the ends and that it is of the same width as the thickness of the wall, and also that the lintel is supported until the cement of the brickwork has set. The height of the wall above the lintel is 6 feet. (*a*) What weight will the transverse strength of the wall itself support, if its safe unit modulus of rupture is 5, and it is considered as a beam with fixed ends? (*b*) What is the minimum depth of the lintel that may be used in order to sustain the portion of the weight that is supported by this member?

Ans. $\begin{cases} (a)\ 5{,}760\text{ lb.} \\ (b)\ 5.6\text{ in.} \end{cases}$

FLAGSTONES

35. Flagstones may be considered according to the arrangement of their supports. When supported at two opposite sides only, they may be analyzed the same as any stone beam of rectangular section. Under unusual circumstances flagstones are supported at two adjacent sides or on three sides, but usually they are supported on all four sides, and under these conditions generally assume structural importance.

SQUARE FLAGSTONES

36. If a square flagstone is supported on four edges it is evident that it will not be so severely strained as when supported at two opposite edges under the same load similarly applied and located. In Fig. 20 is shown a square flagstone, supported on four sides, that is assumed to sustain a uniformly distributed load. The points C_1, C_2, C_3, and C_4 are the centers of gravity of the four divisions of the total load and R_1, R_2, R_3, and R_4 the reactions corresponding with the

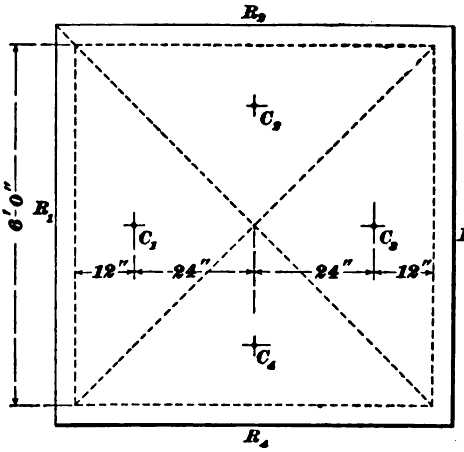


FIG. 20

four equal divisions of the load, which act along the entire lengths of the edges.

37. Bending Moment.—The transverse stress on the flagstone is reduced by the increased number of supports, the additional supports having the common tendency of shortening the span. Exact formulas for finding the maximum **bending moment** on flagstones and their resistance to the stresses created by the load on them have not been determined, and it is doubtful whether formulas founded on mere assumption and hypothesis, without reference to exhaustive tests and practical investigations, will give results

that even approach accuracy. When, therefore, the engineer encounters problems in design, in which the strength of flagstones is required, assumptions must be made that will give results that err on the side of safety.

In Fig. 21 (*a*) is shown the probable method of failure of a square flagstone of homogeneous material supported on its four edges and uniformly loaded, provided that the lines of fracture were first developed along the diagonals, as shown from *a* to *c*. From the assumption regarding the lines of fracture or rupture illustrated at (*a*), the theoretical conditions designated at (*b*) may be analyzed. In the analysis it is assumed that each of the triangular portions *a b c* is a cantilever and that one-fourth of the load is assumed to act about the line of fracture *y y* with a lever arm equal to the distance from the center of gravity of the triangle to the line of fracture *y y*. The distance from the edge of the supporting wall to the center of gravity of the triangle at c_1 is equal to one-third the altitude of the triangular section, or one-sixth of the span of the flagstone; so that the leverage of $\frac{W}{4}$ about the line of fracture is equal to one-sixth of the span of the flagstone, or $\frac{l}{6}$. Therefore, the assumed

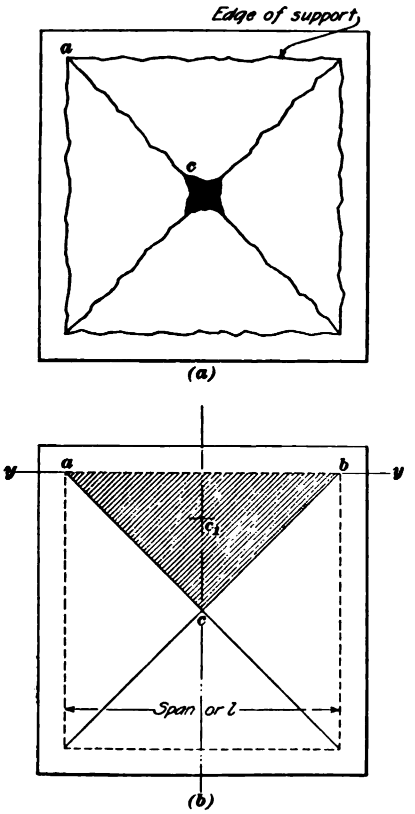


FIG. 21

maximum bending moment may be found by the formula

$$M = \frac{W l}{24} \qquad (12)$$

in which M = bending moment, in inch-pounds;
W = uniformly distributed load, in pounds;
l = clear span of flagstone, in inches.

This analysis shows that a flagstone supported on four sides is subjected to one-third the bending moment of a simple beam uniformly loaded.

38. Resisting Moment.—On the assumptions from which the maximum bending moment was determined, the line of fracture is located at the edge of the supporting wall. Referring to Fig. 21 (*b*), it will be observed that the distance $a b$ is equal to Z, and the distance expressed by the value Z is the breadth of the resisting section, so that if the depth of the flag or the thickness is represented by d, the section modulus of the flagstone will be obtained by the usual formula for rectangular sections, and the safe **resisting moment** is found by the formula

$$M_1 = \frac{l d^2 s_a}{6} \qquad (13)$$

In this formula, s_a represents the safe unit stress.

39. Safe Unit Load.—The flagstone, in order to safely resist the bending moment created by the uniform load, must have a resisting moment equal to the bending moment. Hence, $M_1 = M$, and as $M_1 = \frac{l d^2 s_a}{6}$ and $M = \frac{W l}{24}$, $\frac{l d^2 s_a}{6} = \frac{W l}{24}$; so that, by transposition and reduction,

$$W = 4 d^2 s_a \qquad (14)$$

It will be observed from this formula that the value l is entirely eliminated and that stones of different horizontal dimensions, but of the same thickness, will support equal total loads. But, while they will support the same uniformly distributed load, the unit load per square foot will not be the

same, from the fact that there is a difference in their areas. For instance, a bluestone flag 6 inches thick and 8 feet square between supports, when a **safe unit stress**, or modulus of rupture, of 150 pounds is assumed, will support a safe uniformly distributed load equal to 21,600 pounds, or 337 pounds per square foot; while a flagstone of the same thickness and material, but under which the supports are 10 feet on each side, will sustain the same total load, but will have a safe unit load of only 21,600 ÷ 100, or 216 pounds. It is advisable in using these formulas, because of the uncertainty of the strength of the materials, to use factors of safety ranging from 15 to 30. Such high factors of safety are used, particularly in large stones, because it is impossible to determine their structural defects.

EXAMPLE 1.—What is the maximum bending moment on a square bluestone slab supported 12 feet on each side and loaded with a uniformly distributed load of 200 pounds per square foot?

SOLUTION.—The area supporting the load is equal to 144 sq. ft. and the total load, in consequence, equals 144 × 200 = 28,800 lb. The value of l, in inches, is 144, so that by substitution in formula **12**,

$$M = 28{,}800 \times \frac{144}{24} = 172{,}800 \text{ in.-lb. Ans.}$$

EXAMPLE 2.—What resisting moment, in inch-pounds, is developed along the assumed line of fracture of the flagstone in the preceding example, provided the stone is 8 inches thick and the safe unit stress is 100?

SOLUTION.—By applying formula **13**,

$$M_1 = \frac{144 \times 8 \times 8 \times 100}{6} = 153{,}600 \text{ in.-lb. Ans.}$$

EXAMPLE 3.—A slate slab 2 inches thick having a safe unit stress of 333 pounds and supported on a frame 4 feet 6 inches square, is used for the landing of a combination of iron and stone stairs. What load per square foot of surface will the slab safely support?

SOLUTION.—By applying formula **14**, and substituting the values given in the problem,

$$W = 4 \times 2 \times 2 \times 333 = 5{,}328 \text{ lb.}$$

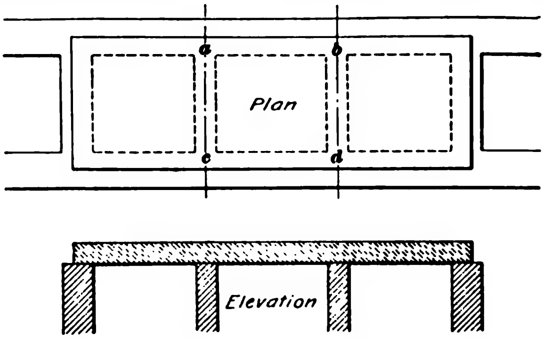
The unsupported area of the slab of slate is equal to 4.5 × 4.5 = 20.25 sq. ft.; hence, the unit load is equal to 5,328 ÷ 20.25 = 263 lb. Ans.

EXAMPLE 4.—A landing slab of slate having a safe unit stress of 180 pounds is 6 feet 8 inches square inside of supports. If the load, inclusive of the weight of the slab, is 200 pounds per square foot, what will be the thickness required to safely support it?

SOLUTION.—By transposition of formula **14**, in which $W' = 4 d^2 s_a$, the value of $d^2 = \frac{W'}{4 s_a}$, and, by substitution, $d = \sqrt{\frac{8,888}{4 \times 180}} = 3.5$ in. Ans.

RECTANGULAR FLAGSTONES

40. Rectangular flagstones continuing over several transverse supports, as shown in Fig. 22 (*a*), may be considered



(*a*)

[redacted]

this arrangement of the supports, the flagstone partakes of the nature of a continuous beam. A theoretical solution of the bending moment that would exist with such an arrangement of supports would be extremely complicated from the fact that, besides being continuous over several supports, the flagstone is sustained along the side edge as well. It is safe, however, to consider a stone of this character as divided into a number of panels, and from the dimensions of a single panel to make the necessary calculations for bending moment, resistance, or load, from formulas **12**, **13**, and **14**.

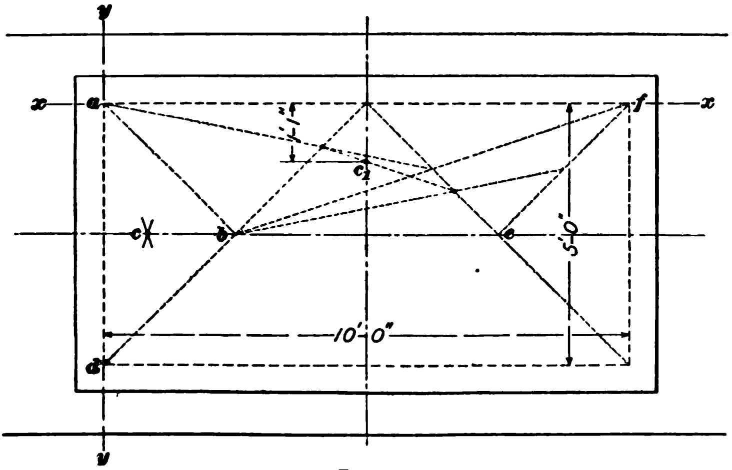


Fig. 23

41. A rectangular flagstone that is supported only along the ends and edges and has no intermediate supports extending transversely, may consistently be proportioned by applying formulas **12**, **13**, and **14**, provided that the longitudinal dimension is not more than one and one-fourth times the transverse dimension. If the length is greater than this, the maximum bending and resisting moment should be determined as suggested in Fig. 22 (*b*). As this figure shows the probable lines of fracture, the bending moment created by the triangular portion of the load about

the line $y\,y$ and the moment of the load on the trapezoidal section of the flag about the axis $x\,x$ should be found. In each case, the load is considered as applied at the center of gravity of the figure, which in the triangle is at the point c and in the trapezoid at the point c_1, and the greatest bending moment is taken along an axis that is located along the inside edge of the supports.

For example, assume that a flagstone is 5 feet by 10 feet along the edge of the supports and that the load per square foot supported by it is 100 pounds. In Fig. 23, which illustrates this problem, the area of the triangle $a\,b\,d$ is equal to 6.25 square feet; consequently, the total load on this portion of the flag is equal to 625 pounds. The moment of this load applied at the point c about the axis $y\,y$ is found, by multiplying its amount by its lever arm of 10 inches, to equal 6,250 inch-pounds. The load on the trapezoid $a\,b\,e\,f$, as found by multiplying its area by the unit load, is equal to 1,875 pounds. The moment of this load acting at the point c_1 about the axis $x\,x$ is determined from the product of $1{,}875 \times 13 = 24{,}375$ inch-pounds. The proportion of the resisting section along the axis $x\,x$ to the resisting section along the axis $y\,y$ is as 2 : 1, while the bending moment about the axis $x\,x$ is about four times that along the axis $y\,y$, so that the flagstone would tend to fail on the axis $x\,x$ rather than along the axis $y\,y$, and its thickness should be so proportioned that the resisting moment and bending moment along the axis $x\,x$ are equal.

42. In calculating the bending moment on flagstones, it is good practice to consider the weight of the stone as part of the load; for instance, if a flagstone is 6 inches thick and weighs 150 pounds per cubic foot, provided that the load safely supported by its resistance to bending is 175 pounds, the allowable unit live load on the flagstone will be equal to 100 pounds.

If a flagstone is laid on solid ground or is supported on a solid masonry foundation and the load is transmitted to it uniformly, there is no bending moment exerted on the stone,

for only a crushing stress is created. Should the load, however, be concentrated, the flagstone must be considered as a bearing block and the bending moments determined as previously explained.

The formulas given for obtaining the assumed safe load of flagstones, whether square or rectangular, supported on four sides, are sufficiently safe for all practical purposes when a factor of safety of from 15 to 20 is employed. There are no formulas by which the strength of flagstones may be calculated that will give exact results and there is hardly any economical necessity for extreme accuracy in the design of such structural elements.

From the fact that small stones are more economical than larger ones, where a considerable area is to be floored, instead of a single slab being used several stones should be employed, so that the end stones will rest on three supports while the intermediate slabs will be simple beams supported at the ends. When the flagstones are arranged in this manner, it is advisable to proportion both the end stones and the intermediates by the formulas for a simple rectangular beam. Long narrow stones, that is, stones where the length exceeds four times the width of the stone, should be analyzed as a series of simple beams placed side by side and having a span equal to the width of the slab, the supports at the ends of the slab being of little assistance on account of the distance separating them.

EXAMPLES FOR PRACTICE

1. Determine the bending moment on a granite slab 10 feet square inside of supports and supporting a uniformly distributed load of 225 pounds per square foot. Ans. 112,500 in.-lb.

2. Find the resisting moment, in inch-pounds, developed along the assumed lines of fracture of a square flagstone 8 feet inside of supports. The stone is 5 inches thick and the safe unit stress is 120 pounds. Ans. 48,000 in.-lb.

3. A square marble platform slab 4 inches thick has a safe unit stress of 80 pounds. What uniformly distributed load will this platform stone safely sustain? Ans. 5,120 lb.

4. What must be the thickness of a bluestone slab used as the floor of a vault 4 feet by 5 feet? The load to be supported is 200 pounds per square foot, the modulus of rupture of the material is 2,700 pounds, and the required factor of safety is 20. Ans. 2.72 in., say, $2\frac{3}{4}$ in.

5. Determine the greatest bending moment on a granite slab 5 feet by 8 feet inside of supports. The load is 180 pounds per square foot of surface. Ans. 31,556 in.-lb.

CORBELS

43. In construction, a **corbel** is any piece of stone, wood, or iron that projects from the vertical face of a wall. Its structural significance is to furnish support for an overhanging wall, or to provide a bearing for an arch, truss, girder, or beam. It may be a single piece of material projecting beyond the face of the wall, or it may be built up of several pieces. When a single piece, the weight that it supports tends to turn the stone on the center of its bearing on the wall with a moment equal to its amount multiplied by the distance from the line of action of the weight to the point of rotation. This turning moment is resisted by the moment of the superimposed weight created by the masonry above the corbel acting on the end that is built in the wall.

The overhanging portion of the corbel may, for convenience, be designated as the *weight arm*, while the portion of the corbel built in the wall may be regarded as the *power arm* of the cantilever. A corbel built of several pieces should have each piece balanced the same as a single cantilever, for the single stones, or elements, in the corbel must have a safe margin of moments on its power arm over the moments of the weights on its weight arm.

If a force acts on a corbel in a direction not at right angles to its bed, the effect is the same as if the corbel were subjected to two separate forces—the horizontal and vertical components of the oblique force. The horizontal force tends to slide the corbel on its bed and the vertical force tends, in effect, to break off the projecting portion. The vertical force, in consequence, produces bending stresses in the material and also vertical shearing stresses.

44. Whether a corbel is of one piece of material or of several, the distance that it enters the wall is determined by the consideration that the resultant reaction on its bed will not lie outside of the middle third of the joint, though some authorities admit that if the line of the resultant is within the middle half, the corbel will retain its position.

This statement is more clearly explained by reference to Fig. 24. Here W_1 equals the load supported on the projection of the corbel, while W_2 equals the weight of the wall above the corbel. These two forces acting downwards must be equalized by an upward reaction, which is represented by R. The amount of this upward reaction, since the algebraic sum of all forces acting vertically must equal zero, is equal to the sum of W_1 and W_2. The amount of W_1 is equal to the end reaction of the beam, girder, or truss supported by the corbel and is a known force, while the amount of the weight W_2 is determined by the distance the corbel is built into the wall and by the height of the wall above the corbel. If, then, the height of the wall and the projection of the corbel into it are insufficient to give the requisite amount of weight, the reaction R will move outwards toward the face of the wall or projecting end of the corbel, and instead of its line of action falling within the middle third of the bed, as represented by $x_1 x_1$, which is one-third of the distance $x x$, it will, in order to balance the forces W_2 and W_1, act outside of the middle third, or in the position shown by the dotted arrow R_1. When the reaction R is thus shifted nearer the edge of the wall, the crushing stress at the edge is usually greater than the supporting masonry will sustain. If, on the other hand, the wall is of sufficient height above the corbel and the corbel has adequate bearing in the wall, the weight W_2 will be of such amount and will be applied at such a position that the

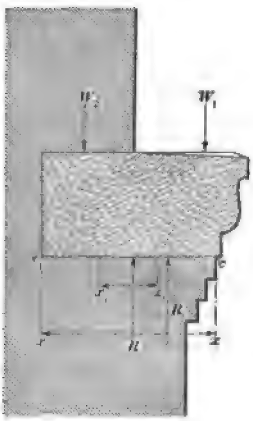


FIG. 24

reaction R will fall within the middle third of the bed of the corbel, so that neither undue pressure will exist at the edge c nor tension be created at the edge c_1.

45. Determining the Stability of a Corbel.—The analysis of the stability of a corbel is usually simple. Before the weight of wall required above the corbel to hold it in position can be determined, the position of the upward reaction from beneath must be assumed; the point of application of this reaction may then be considered as the center of moments around which the load supported by the corbel and the weight of the superimposed masonry tend to

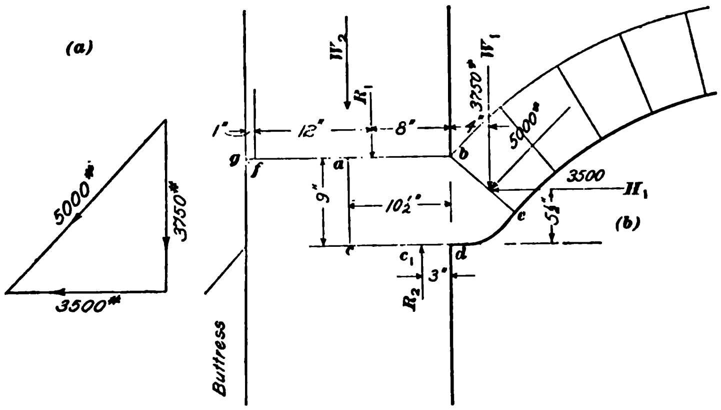


FIG. 25

rotate. In order that the several steps in the analysis of a corbel may be made clear, the following examples have been assumed and demonstrated.

Suppose that it is desired to determine if the corbel *abcde*, Fig. 25 (*b*), is sufficiently stable to sustain the thrust of the arch ring it supports. As an oblique thrust of 5,000 pounds is applied at a point within the middle third of the joint *bc* and acts in a direction perpendicular to the plane of the joint, to determine the stability of the corbel, this thrust must first be resolved into its horizontal and vertical components, which may be accomplished by drawing the triangle of forces shown at (*a*). The two forces thus found

consist of a vertical force of 3,750 pounds and a horizontal force of 3,500 pounds, and both at the point of application of the oblique thrust. The vertical force tends to overturn the corbel about the point d, in which rotary action the point a is raised and the superimposed wall is inclined to tilt or turn about g. The horizontal force acts in a manner to shear the corbel and wall along the lines of the bed or joint.

The corbel is retained in equilibrium of rotation by four forces—W_1, R_1, R_2, and H_1. Owing to the fact that the supporting masonry is likely to chip and spall at the point d, the effective joint $c\,d$ is taken at $\frac{1}{2}$ inch inside of the vertical face of the wall, so that the length of the corbel bed is 10 inches. As previously stated, the point of application of R_2 should fall within the middle third or half of the joint. In this case, it is practical to allow the reaction R_2 to be applied at the extreme edge of the middle half of the bed, which will make its point of application c_1 $2\frac{1}{2}$ inches from the edge of the effective joint, or 3 inches from the vertical face of the wall. Around this point the force W_1, acting with its lever arm, tends to rotate the corbel in one direction, while the forces R_1 and H_1 tend to turn it in the opposite direction.

The amount of R_1 is determined by dividing the difference between the moment of W_1 about the point c_1, and the moment of H_1 about the same point by the distance of the line of action of R_1 from this point. Thus,

$$\begin{aligned} \text{Moment of } W_1 &= 3{,}750 \times 7 &= 26250 \text{ inch-pounds} \\ \text{Moment of } H_1 &= 3{,}500 \times 5\tfrac{1}{2} &= 19250 \text{ inch-pounds} \\ \text{Difference of moments} &= &7000 \text{ inch-pounds} \end{aligned}$$

This difference, 7,000, divided by the leverage of R_1, or 5 inches, gives 1,400 pounds as the amount of R_1. This demonstration proves that with a load or weight equal to 1,400 pounds applied at R_1, the corbel will be in equilibrium of rotation; that is, there will be no tendency for it to revolve about the point c_1 and in this way be displaced. The position of R_1 is assumed at the extreme edge of the middle half of the bearing $a\,b$ in the same manner as the position of R_2 was determined. The location of this reaction R_1 was taken at

the edge of the middle half away from the inside wall, because by this means the extreme leverage about c_1 was obtained. If it had been assumed at the center of the upper joint, a greater weight would have been required. The amount of the reaction R_2 must equal the sum of R_1 and W_1, or 5,150 pounds, since the algebraic sum of all the forces acting vertically on a body in equilibrium must equal zero.

This analysis shows that the corbel is safe when a weight of wall amounting to 1,400 pounds is concentrated at R_1.

46. There is a possibility of the superimposed portion of the wall above the corbel tilting about the edge of the wall opposite the corbel. In overturning, the point of revolution of the wall will, theoretically, be located at the edge g, but if the wall revolves about the edge g, the masonry will crumble away until sufficient area is obtained to carry the weight at this point. The center of moments for the superimposed wall should then be taken some distance in from

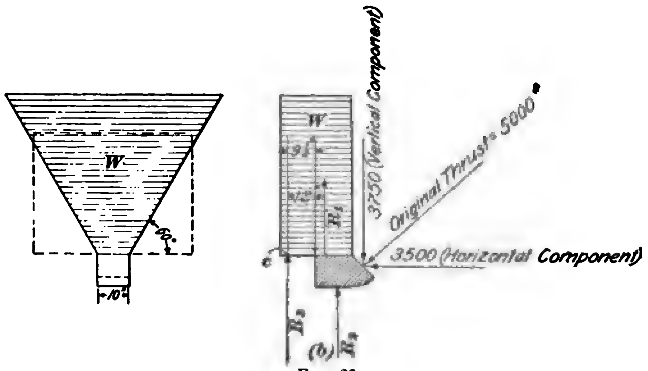


FIG. 26

the face, assumed in this case to be 1 inch. It is assumed that the wall shown in the figure is 7 feet high above gb. Therefore, in order that the corbel may turn the superimposed wall about the point f, it must lift the triangular-shaped piece of wall W, as shown in the elevation, Fig. 26 (a). The face of the portion of the wall acting on the upper bed of the corbel is assumed to be a triangle for the reason explained under Lintels. The load of masonry

to be lifted by the tilting of the corbel is then equal to the weight of the triangular piece of masonry or about 7,600 pounds and the conditions that exist are shown in Fig. 26 (*b*). In this diagrammatical figure, W acts downwards at the center of the wall and with a lever arm about the point c equal to $9\frac{1}{2}$ inches, while the upward force at the end of the corbel tending to lift the wall, which is equivalent to the reaction R_1, Fig. 25, has a leverage about the point c equal to 12 inches. If the moment of the superimposed wall about the point c is regarded as positive and the moment of the lifting tendency at the corbel is considered as negative, the positive and negative moments are as follows:

$$\text{Positive moment} = 7{,}600 \times 9\tfrac{1}{2} = 72{,}200 \text{ inch-pounds}$$
$$\text{Negative moment} = 1{,}400 \times 12 = 16{,}800 \text{ inch-pounds}$$

A comparison of these moments shows that there is ample resistance against the superimposed wall overturning by the tilting effect of the corbel. The wall will have an additional factor of safety against tilting, from the fact that the adhesive strength of the mortar along the line of fracture will amount to considerable and cause increased resistance.

47. The resistance to crushing, of the masonry beneath and above the corbel, should next be investigated. The bearing strength of the masonry is governed by the resistance of the mortar to crushing; this for Portland cement is 200 pounds per square inch. The joint *a b*, Fig. 25 (*b*), is acted on by the force R_1 of 1,400 pounds applied at a distance of $2\frac{1}{2}$ inches from the edge. Then, by applying formula **1**, the maximum pressure k_1 per lineal inch may be determined. By substitution,

$$k_1 = \frac{2 \times 1{,}400\,(2 \times 10 - 3 \times 2\frac{1}{2})}{10 \times 10} = 350 \text{ pounds}$$

This pressure is distributed over a width of block equal to 10 inches, as shown in Fig. 26 (*a*), so that the maximum pressure per square inch at the extreme edge of the corbel is equal to $350 \div 10 = 35$ pounds. This calculation shows that the maximum pressure on the upper bed of the corbel

of the masonry is well within the allowable bearing value. The maximum pressure, instead of being on the bed *a b*, might exist at the joint *c d*. Here

$$k_1 = \frac{2 \times 5{,}150\,(2 \times 10 - 3 \times 2\tfrac{1}{2})}{10 \times 10} = 1{,}287.5 \text{ pounds}$$

The width of the joint being 10 inches, the pressure per square inch at the edge will equal 1,287.5 ÷ 10 = 128 pounds. The latter result, which is the maximum pressure on the bottom bed of the corbel, exceeds the former result of maximum pressure on the top bed of the corbel, but is, nevertheless, well within the bearing value of the masonry.

48. The corbel may now be analyzed for resistance to bending. Because the horizontal component of the oblique thrust of the arch acts so nearly coincident with the neutral axis of the rectangular section of the corbel, only the vertical forces acting on the corbel will be considered in determining the bending moment. The vertical forces acting downwards are R_1 and the weight W_1, while the opposing force, which is equal to the sum of these two, is R_2. The theoretical bending moment on the corbel is the moment of either W_1 or R_1 about the point of application of R_2. Considering the load W_1, the bending moment will equal 3,750 × 7 = 26,250 inch-pounds. The section modulus of the rectangular section, or S, is equal to $\frac{b\,d^2}{6}$. Whereupon, by substituting the values of b and d, $S = \frac{10 \times 9 \times 9}{6} = 135$. Therefore, if the modulus of rupture of the material is 1,500 pounds, the stone in this case being considered as limestone, and a factor of safety of 10 is required, the safe unit stress will equal 150 pounds. In consequence, the safe resistance of the corbel to transverse stress, or M_1, will equal 135 × 150 = 20,250 inch-pounds. This result compared with the bending moment of 26,250 inch-pounds previously obtained shows a deficiency, but because it is securely built in the masonry, which tends to materially reduce the bending stresses, the corbel may be considered safe.

49. The final analysis to which the corbel may be subjected is that of shear along the vertical face of the wall. The vertical shear tending to cut off the portion of the corbel marked *b d e*, Fig. 25 (*b*), is equal to the weight W_1, or 3,750 pounds. The area subjected to shearing stress is 10 inches × 9 inches = 90 square inches, so that if the safe unit shear of limestone is taken at 100 pounds, the shearing resistance will equal 9,000 pounds. This result is in excess of the required resistance.

50. In solving this problem, it was necessary to consider the several ways in which the corbel would tend to fail. For convenience in analyzing the stability of a corbel, the following arrangement of investigations may be adopted:

1. Assume the upward reaction on the bed of the corbel, or R_2, Fig. 25 (*b*), to be located within the middle third or half of the bed, and determine what will be the amount of the downward reaction, or R_1, on the upper bed of the corbel when this force is likewise located at an assumed point within the middle third or half of the bed. Also, calculate the possible weight of the superimposed wall on the upper bed of the corbel, in order to determine whether it is equal to the amount required for the reaction R_1.

2. From the amounts of R_1 and R_2, or the downward and upward reactions acting, respectively, on the upper and lower beds of the corbel, determine what will be the maximum unit pressure on the masonry at the upper and lower edges of the corbel. Also, investigate the bearing strength of the masonry in order to determine whether it has a sufficient safe unit resistance to crushing to equal the maximum unit pressures obtained at the upper and lower edges.

3. Determine the maximum bending stress on the corbel created by the downward and upward vertical forces acting on its upper and lower beds, considering these loads and reactions as concentrated forces.

4. Determine whether the corbel has sufficient shearing resistance along the section cut by the vertical plane of the wall from which it projects.

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51. In order to illustrate further the application of the principles of analysis of corbels, attention is directed to Fig. 27. The corbel shown is made of several pieces of limestone, and supports an external load of 10,000 pounds uniformly distributed. The total projection of the corbel is 18 inches and the width is 24 inches, while it is built into a 17-inch wall the full depth.

The first step is to figure the required reactions, considering the corbel as made of one piece. If the reaction acting upwards on the bottom bed of the corbel must lie within the middle half of the joint, the distance from the point of application of the upward reaction to the edge of the wall must be at least $\frac{17}{4}$, or $4\frac{1}{4}$ inches. The required downward reaction may be found by considering the point of application of the upward reaction as the center of moments, at the right of which the 10,000-pound load on the corbel acts at a distance of $13\frac{1}{4}$ inches. This load is to be balanced by the weight

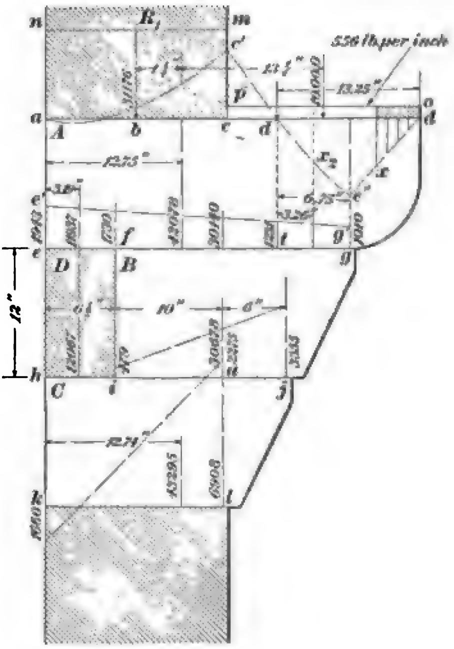

FIG. 27

of the superimposed wall acting at a distance of $4\frac{1}{4}$ inches, as shown in the illustration. The weight of the wall must be found from the equation:

$$R_1 = 10{,}000 \times \frac{13.25}{4.25} = 31{,}176 \text{ pounds}$$

In determining the amount of R_1, its location was taken

at the center of the upper bed of the corbel, owing to the fact that the corbel extends through the wall.

The pressure on the bed joint $e\,g$ of the stone A is the sum of 10,000 pounds, 31,176 pounds, and the weight of the stone, amounting to about 902 pounds. This combined weight is equal to 42,078 pounds and is the amount of the upward reaction at the lower bed of the corbel.

52. The next step is to consider the pressure on the joint of each individual stone. The maximum pressure per lineal inch on the joint eg is found, from formula **1**, by substituting the numerical values for the several symbols in the equation. For this problem, $l = 28.5$, $l_1 = 12.75$, $l_2 = 15.75$, while $P = 42{,}078$, and, by substitution, k_1 is found to be 1,943 pounds and k_2 1,010 pounds; this result can be shown graphically by the trapezoid $e\,e'\,g'\,g$, which represents the variation of pressure on the joint $e\,g$.

To find the maximum pressure on the upper joint of the stone B, it is necessary to scale the ordinate of the trapezoid $e\,e'\,g'\,g$ directly above the edge f of this stone. The amount of pressure at this point on the bed joint eg is found to equal, approximately, 1,730 pounds. Since the minimum and maximum pressures at the extreme edges of the stone B have, in this manner, been determined, the average pressure may be found, and the total pressure, or load, on the stone will be equal to the product of the average pressure by the length of the joint, or 22 inches, which is found to be equal to 30,140 pounds. In a similar manner, the load on the stone D is calculated to be 11,937 pounds.

The distance that this total load of 30,140 pounds is located from the edges f and g of the stone B may be found by applying formulas a and b. The values to be substituted in these formulas are, $l = 22$, $k_1 = 1{,}730$, and $k_2 = 1{,}010$; hence,

$$l_1 = \frac{1{,}730 + 2 \times 1{,}010}{1{,}730 + 1{,}010} \times \frac{22}{3} = 10.04 \text{ inches}$$

$$l_2 = \frac{1{,}010 + 2 \times 1{,}730}{1{,}730 + 1{,}010} \times \frac{22}{3} = 11.96 \text{ inches}$$

Successively, as above, the distances from the right- and left-hand edges are found to be equal to 10.04 and 11.96 inches, and the sum of these dimensions equals the total length of the joint, which makes an excellent check on the calculations.

The pressure on the joint ij is 30,140 pounds plus the weight of the stone B, which, in this case, is found to equal, approximately, 538 pounds, making the total pressure 30,678 pounds. By again employing formula **1**, the maximum and minimum pressures on the joint ij are found to equal 3,355 and 479 pounds, respectively. The unit pressures at the extreme edges may be found by dividing these pressures by the allowable width of the joint, or $24 - 1 = 23$ inches, as $\frac{1}{2}$ inch has to be deducted from each side. In this instance, $3{,}355 \div 23 = 146$ pounds and $479 \div 23 = 20$ pounds, which results represent the pressure at the edges per square inch of bearing surface.

The weight of the piece of brickwork D, allowing 120 pounds per cubic foot, is 130 pounds, and the pressure brought to bear by the stone A is 11,937 pounds. These added give a total pressure of 12,067 pounds on the joint hi. It is observed that two loads act on the stone C, one of 12,067 pounds from the brickwork D, and one of 30,678 pounds from the stone B, the total load thus being 42,745 pounds. By moments, the reaction of these loads is found to act at a distance of 12.74 inches from the inside face of the wall. The weight of stone C is 550 pounds, so that the total load on the joint kl is 43,295 pounds. By formulas **1** and **2**, respectively, the maximum pressure is found to be 6,908 and the minimum, 1,660 pounds, estimating the joint to be 16.5 inches long. Inspection of the diagram shows that the maximum pressure per lineal inch is at l of the joint kl. This maximum pressure divided by 23, the allowable width of the joint in inches, after deducting $\frac{1}{2}$ inch from each side, is 300 pounds per square inch, the pressure at k being minus 72 pounds. This indicates that mortar of the best Portland cement must be used in setting these corbel stones.

53. By inspection of Fig. 27, it is evident that the greatest bending moment is produced in the stone *A*, because it is the longest, and the several forces act with the greatest lever arms. It is unnecessary, therefore, to consider the bending moment in the other stones, for if the stone *A* is safe against failure by transverse stress, the other stones, being of the same thickness and width, will evidently possess sufficient strength. In proceeding with the determination of the greatest bending moment, it is well to find a line that will represent the variation of shear produced in the stone *A*. To find such a line, all that is necessary is to calculate the algebraic sum of the loads represented by the rectangles *a c m n*, *c d o p*, and the trapezoid *e g g′ e′*. The line, showing the variation of the shear, is designated by *a c′ d′ e″ d*, and may be described by commencing at the point *d* and laying off ordinates to scale at points some unit of length apart, these ordinates being made, by scale, to equal the amount of the pressure from the point *d* to the ordinate in question. For instance, the length of the ordinate x is by any scale equal to the total pressure of the shaded portion of the rectangle *c d o p*. The length of the ordinate x_1 is equal to the total amount of the pressure on the upper surface of the corbel from this ordinate to the point *d*, minus the amount of pressure on the bottom bed from this ordinate to the point *g*. In a similar manner, all the points in the curve may be obtained. It is a question whether the corbel acts as a simple beam or as a cantilever, and, therefore, whether the greatest bending moment on the stone *A* will lie at the point of greatest shear or at the point of no shear. In this instance the greatest bending moment is found at the point where the shear changes sign, that is, where the line showing the variation of shear crosses the horizontal line *a d*. This point of no shear is designated in the diagram by *d′*; the bending moment about the point *d′* may be found from the formula

$$M = P_1 l_2 - P l_1 \qquad (15)$$

in which P_1 = load or pressure on upper surface of corbel existing on that portion between points d' and d;

P = total pressure on lower surface of stone between points t and g;

l_2 = distance from center of action of pressure P_1 to point d';

l_1 = distance from center of action, or point of application of pressure P, to a vertical line passing through point d'.

The amount of the pressure P_1 is equal to $556 \times 13.25 = 7{,}367$ pounds, while the amount of P is equal to the mean pressure between t and g multiplied by the distance tg. Consequently, $P = \frac{1{,}010 + 1{,}231}{2} \times 6.75 = 7{,}563$. The dimension l_2 is equal to one-half of the distance from d' to d, or 6.625, while l_1, which may be obtained by applying formula **a**, is found to equal 3.26. Then, by substituting in the formula for the bending moment,

$$M = (7{,}367 \times 6.625) - (7{,}563 \times 3.26) = 24{,}151 \text{ inch-pounds}$$

When a safe unit stress of 150 is assumed, it may be considered that the section of the corbel stone A is ample to resist the transverse stress about the vertical line passing through the point d'.

54. In analyzing the shearing stress on the corbel, it is seen by inspection that the greatest load exists on the pro-

are common to all structural engineering problems. It is unnecessary in small corbels where a comparatively light load is sustained, to employ these calculations and diagrams, but where the corbel is of great structural importance, it will be well to follow the method of analysis given, in order to be absolutely sure of the stability of the corbel.

EXAMPLES OF CORBEL CONSTRUCTION

56. In Fig. 28 is shown a detail of a corbel-supported bay window. The wall is built to the level of the bottom bed of the stone *A*, and the stone *B* is set on a $\frac{1}{4}'' \times 4''$ wrought-iron bar *c* laid on top of the wall. This bar, which runs the length of the wall, gives a positive bearing for the stone and has greater strength than a cement joint. By its

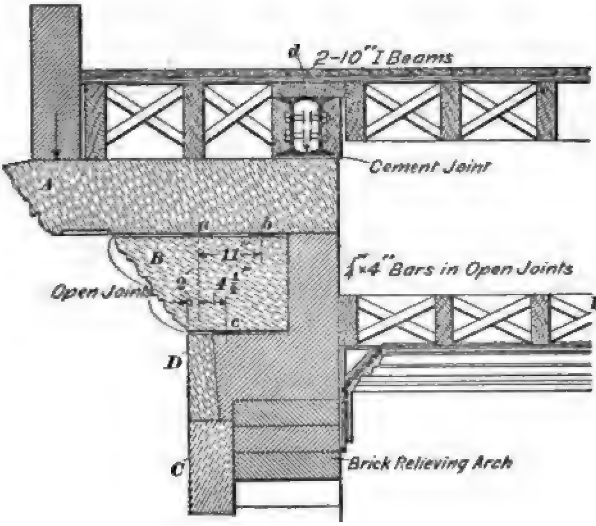

FIG. 28

use the center of pressure is accurately located, which is an advantage in calculating the forces acting on the corbel. The stone *A* rests on two such wrought-iron bars *a* and *b*, and as it is a cantilever, it is prevented from tilting by the two **I** beams at *d*, the ends of which are built securely into the wall. It will be noticed that the bar *c* is placed on

the brickwork and not on the stone *D*. By observing this

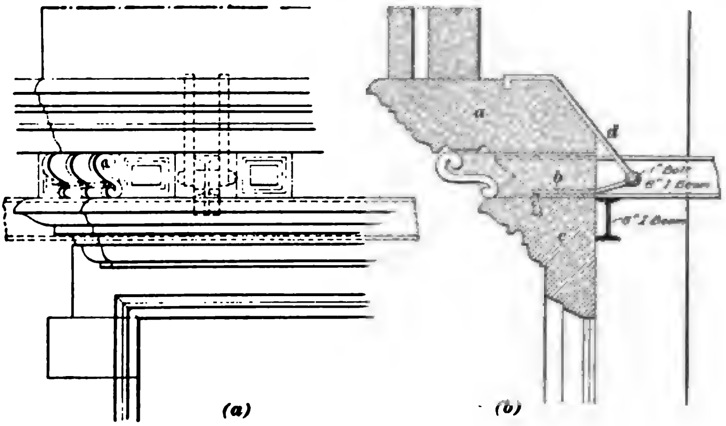

FIG. 29

precaution, there is no tendency to force the shallow-faced stones, or ashlar work, out of place.

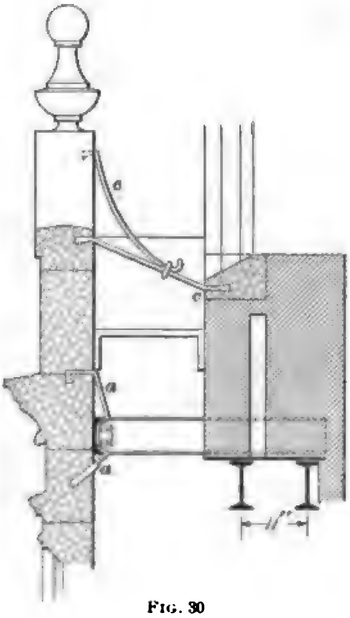

FIG. 30

In Fig. 29 (*a*) and (*b*) is shown another interesting example of construction. An 8-inch **I** beam is built into the wall of the building and supports on its projecting end the corbel stone *a*. This stone is prevented from tilting by the anchor bar *d*, which is firmly secured to the **I** beam in a manner more clearly shown in (*a*). In the detail of construction shown, the stone *a* acts as a corbel and the 8-inch **I** beam as a cantilever. The stone *c*, which is somewhat overbalanced by its projection, is also anchored to the cantilever beam. The ends of the anchors, in order that they may be

more securely connected with the stones, are made dovetailed and are set in the stone with lead.

In the details shown in Figs. 28 and 29, open joints are left at the edge of the corbel stones in order that there may be no danger of undue pressure spalling the fine carved work beneath. After the building has been constructed and all parts have taken their bearing, these joints may be pointed.

In Fig. 30 is shown a method of constructing and anchoring the face wall of a bay window. Here, two supporting I beams carry cantilever beams, the projecting ends of which are secured to a bent channel corresponding in plan with the outline of the window. To this channel the several stones are securely anchored, as shown at *a*, *a*. The cantilever carries very little weight, its function being to act as a support for the channel *b*, which provides the necessary holdings for the anchors. The finial is well anchored in several places by the construction shown at *c*, *c*.

STATICS OF MASONRY

(PART 2)

FOUNDATIONS

1. The design of **foundations** is one of the most important studies in connection with building construction, because the entire stability of the structure depends on them, and their proper proportioning prevents unequal settlement and eliminates its unfortunate consequence. The principal condition to attain in proportioning foundation **footings**, as the spread at the base of a foundation wall or pier is called, is to have the pressure on the earth or soil well within its safe unit bearing value, and also, what is of equal importance, to have the pressure on the soil approximately uniform. The factors then that are to be considered in the design of foundations are the weight from the superimposed load and the bearing capacity or sustaining value of the soil. This latter is explained in *Materials of Structural Engineering*, Part 2, under the heading Foundation Soils, where the bearing values for different soils are given in a table.

THE LOAD ON FOUNDATIONS

2. The floor loads consist of the dead and live loads; therefore, the joists, beams, or girders supporting a floor or roof rest on a wall or are supported directly or indirectly by a column that rests on a wall or footing. The dead load due to the weight of the floor must be considered in proportioning the footings. It is not usual in designing the

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footings, however, to figure on any of the live loads except where the building is erected specifically for storage purposes, such as warehouses for merchandise. In all other buildings, it is considered the best practice to proportion the footings for the full dead load only, but to keep the pressure on the soil well within the safe bearing value. If the live load were considered in proportioning the footings, the ratio between the live and dead loads would not be the same for the several columns and walls, so that unequal settlement would take place on account of variations in the amount of the live loads.

This is more clearly explained by assuming such a condition of loading as might occur in an office building. For instance, the inside columns, that is, those located within the interior of the building, sustain a dead load of 75 tons and a live load of 50 tons, while the exterior column at the corner of the building supports a dead load due to the floors and exterior walls equal to 200 tons and a live load of 25 tons. The live load may be realized or not; at least there will be great variations in its amount, depending on the conditions that obtain. If the foundation for the interior column is proportioned for both the dead and the live loads, and the soil is assumed to have a maximum safe bearing value of 6 tons per square foot, the area of the footing for this column will be $125 \div 6 = 21$ square feet, approximately, while the footings under the exterior column will have an area of $225 \div 6 = 37\frac{1}{2}$ square feet. It is evident that on the interior column a greater percentage of the entire load is live and variable than on the exterior column. Hence, if the same footing area were maintained and the live load reduced to nothing, which is likely to occur, the footing under the interior column would exert a unit pressure on the soil equal only to $75 \div 21 = 3.57$ tons, while the pressure on the soil under the exterior corner column would equal $200 \div 37\frac{1}{2} = 5.33$ tons per square foot. From this it will be observed that when the footings are proportioned to include both the fixed and the variable load, on the reduction of the variable, a great inequality of pressure on the soil will exist

throughout the several footings and unequal settlement is likely to take place, provided that the maximum allowable pressure on the soil was assumed.

3. The proper proportioning of foundation footings depends greatly on the judgment and experience of the designer. The result that it is desired to attain is that the settlement of the building shall be reduced to a minimum and that it shall be uniform. The footings should be designed to sustain the average or usual load.

If the live load is variable, it is frequently disregarded, as explained in Art. **2**, for where a load is intermittent and considerably less than the dead load it is likely to have little effect on the soil and will cause no appreciable settlement of the building. The dead load may be figured to a certainty and is the same at all times. The snow load on roofs may generally be disregarded entirely in proportioning foundation footings, as may likewise be the wind load, for these loads are intermittent, and as the soil under the footing has become compact from the dead load, they are unlikely to cause any settlement.

There is some disagreement among engineers as to whether it is advisable to neglect the entire live loads in designing the footings in office or other important buildings, and very conservative practice recommends that in all cases a certain percentage of the live load should be assumed in proportioning the footings. The percentage is to be determined by the designer, and is essentially influenced by the nature of the soil and the purpose for which the building is to be used. The practice generally recommended, when a percentage of the live load is considered, is to proportion the footings, for buildings of more than three stories in height, to sustain loads computed as follows, the full dead load being always considered:

Warehouses and factories should have the footings designed to sustain the full live load.

Stores and buildings for light manufacturing purposes, churches, schoolhouses, theaters, and halls for public assembly

should have the footings proportioned for 75 per cent. of the live load.

Office buildings, hotels, and apartment houses should have footings designed for 50 per cent. of the live load.

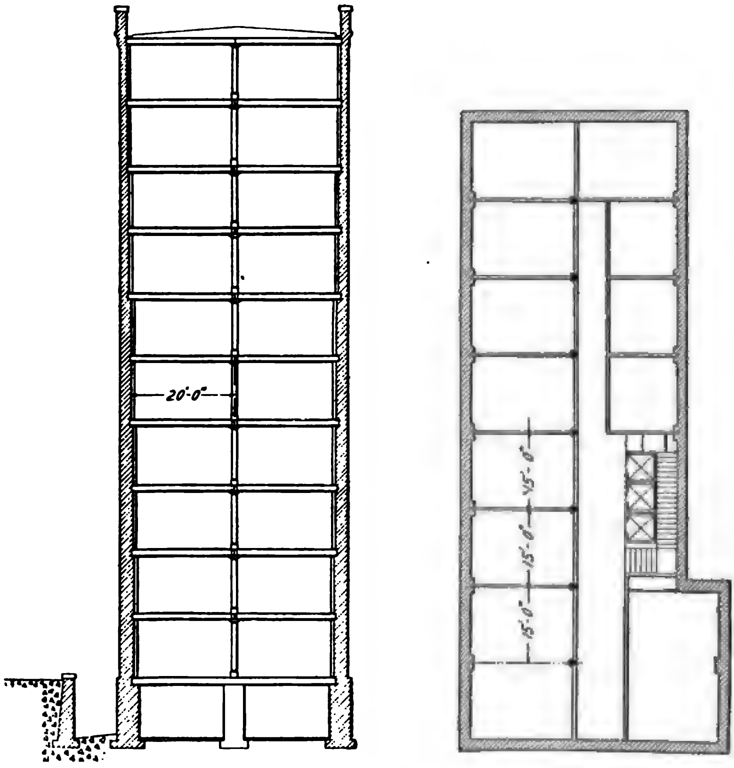


FIG. 1

EXAMPLE.—What should be the width of the footings under the outside walls, and what should be the area of the footings under the piers supporting the columns in the office building shown in Fig. 1, provided that the soil is capable of carrying safely 4 tons per square foot? The live load on each floor is 80 pounds per square foot, inclusive of partitions, and the dead load of each floor is 60 pounds per square foot, which includes the weight of the columns. The weight of the roof, including its snow load, is taken at 25 pounds per square foot, while

the weight of the exterior wall per running foot has been calculated to be 30,890 pounds.

SOLUTION.—Having the dead load on the footings due to the walls, it is necessary to figure the live load that is transmitted from each floor by the beams to the wall. There are ten floors, and from the plan shown, the floor area supported by each lineal foot of wall is approximately equal to one-half the span of the floor beams, or $\frac{20}{2} \times 1 = 10$ sq. ft., so that the entire floor area supported by 1 ft. of wall is $10 \times 10 = 100$ sq. ft. The entire live load on 1 ft. of wall consequently is $100 \times 80 = 8,000$ lb.

According to Art. **3**, only 50 per cent., or 4,000 lb., of this load need be figured in proportioning the footings. To this must be added the roof load for 1 ft. of wall, which is, approximately, $25 \times 10 \times 1 = 250$ lb., making the total weight on 1 ft. of wall equal to:

Dead load	30890 lb.
50 per cent. of live load	4000 lb.
Roof load	250 lb.
Total	35140 lb.

The weight, therefore, on each lineal foot of wall, in tons, is practically equal to 18, so that if the soil, as stipulated in the problem, will safely sustain 4 tons per square foot, the width of the footing must be $18 \div 4 = 4$ ft. 6 in. Ans.

The size of the footings under the piers supporting the columns is found as explained hereafter in Art. **6**.

DESIGN OF FOUNDATIONS

4. The bearing capacity of the masonry in foundation walls and piers must be ascertained, as well as the distribution of the pressure on the soil. In order that the resistance of a foundation wall or pier to crushing may be determined, the resistance per unit of surface measurement, as the square foot or square inch, must be known. The ultimate and allowable unit bearing values that have been obtained through tests and by experience are given in *Materials of Structural Engineering*, Part 2, and these values represent the best current practice. Not only must the requisite area at the base of a foundation footing be determined, but the several sections of a foundation wall or pier must be analyzed to see whether the pressu masonry is well within its allowable resistance to

This is more clearly explained by reference to Fig. 2 (*a*) and (*b*). In (*a*) is shown a brick wall resting on a stone foundation wall that, in turn, is supported by a concrete footing course, while in (*b*) is shown the elevation of a column pier. This shows a cast-iron column base resting on a granite cap that is sustained by a brick pier resting on a stone footing.

In the design of the foundation wall shown in Fig. 2 (*a*), it is first necessary to determine whether the brickwork

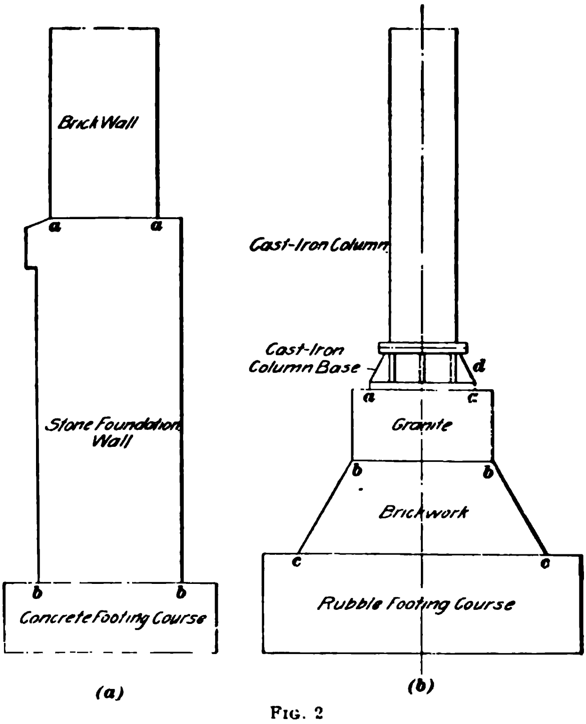


FIG. 2

at *a a* is of such a width as to prevent the crushing of the stone wall beneath, and it is likewise necessary to ascertain whether the stone wall has sufficient width at *b b* to preclude any possible failure of the concrete by crushing. In the column pier shown in Fig. 2 (*b*), the

cast-iron column must be so proportioned that its section will offer sufficient bearing area and, in turn, the bottom flange *a a* of the cast-iron base must have such an area that the allowable bearing value of the granite capstone will not be exceeded. The bottom surface of the capstone *b b* must have such an area that the safe bearing value of the brickwork is not exceeded, and the surface of the brickwork at the bottom of the pier *c c* must be so proportioned as to create a pressure on the stonework forming the footing, within the allowable compressive strength of rubble masonry. In each, the footing course must have sufficient bearing area on the foundation soil to sustain the superimposed load.

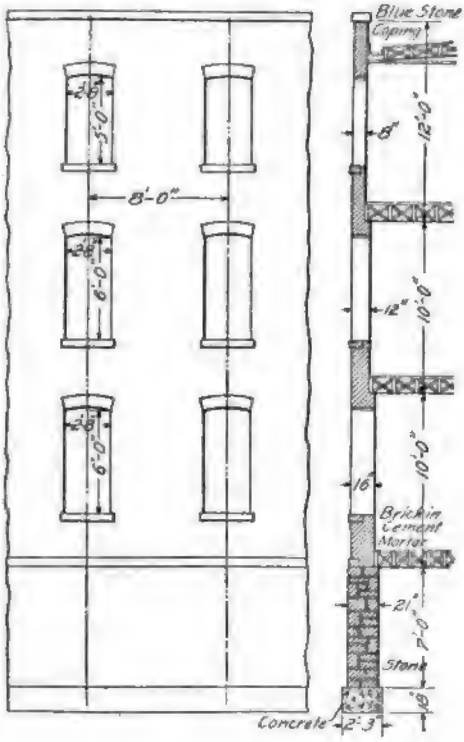


FIG. 3

PROPORTIONING FOUNDATION FOOTINGS

5. In calculating the weight on the footings under any masonry wall, it is necessary to figure the weight on the wall for each lineal foot, and then the width of the footing may readily be determined by dividing by the safe unit bearing value of the soil.

EXAMPLE.—The soil under the foundation footings of the building shown in Fig. 3 is a poor sandy clay, capable of sustaining with safety

a load per square foot of only 1½ tons. What will be the width of the footing, provided that the window sash, frame, and glass weigh 150 pounds for each window?

SOLUTION.—None of the floors nor any portion of the roof is supported on the foundation, as the floorbeam joists extend parallel with the walls and not at right angles to them, so that the only load on the footing is due to the weight of the brick wall, the stone foundation wall, and the concrete footing. The weight per cubic foot of brickwork laid in cement mortar is taken at 130 lb., and the weight of rubble masonry and concrete is approximately equal to 150 lb. per cu. ft., while the weight of bluestone is taken at 160 lb. In figuring the weight of the concrete footing course, which forms a portion of the weight on the soil, it is necessary to assume the width of the footing; in this case the assumed width is 27 in. The weight, in pounds, without deducting for the window openings of the several sections of the wall and the foundation included in 8 lineal ft., which is the distance from center to center of window openings, may be calculated as in the following tabulation:

SECTIONS	POUNDS
Footings, 1.5 × 2.25 × 8 @ 150 lb. per cu. ft.	4050
Foundation, 1.75 × 7 × 8 @ 150 lb. per cu. ft.	14700
Brick wall, first story, 1.333 × 10 × 8 @ 130 lb. per cu. ft. . .	13863
Brick wall, second story, 1 × 10 × 8 @ 130 lb. per cu. ft. . .	10400
Brick wall, third story, .666 × 12 × 8 @ 130 lb. per cu. ft. . .	8312
Coping course, .5 × 1 × 8 @ 160 lb. per cu. ft.	640
Total weight, without deducting for window openings . .	51965

The existence of window openings in the wall diminishes the weight on the footings, and the weight of the masonry omitted in forming them should be deducted. The weight of this masonry is as follows:

	POUNDS
One window opening, 2.666 × 6 × 1.333 @ 130 lb. per cu. ft. .	2772
One window opening, 2.666 × 6 × 1 @ 130 lb. per cu. ft. . . .	2080
One window opening, 2.666 × 5 × .666 @ 130 lb. per cu. ft. . .	1153
Total amount of masonry to be deducted for window openings	6005

Deducting this amount, 6,005 lb. from 51,965 lb. gives 45,960 lb., the net weight of the masonry in the wall, and to which must be added 450 lb. for the weight of three window sashes and frames, as stated in the problem. This makes the entire weight on 8 lineal ft. of footing 45,960 + 450 = 46,410 lb. The weight on 1 lineal ft. of the footing is, in consequence, 46,410 ÷ 8 = 5,801 lb. The theoretical width of the footing, from the information given, is obtained by dividing the entire pressure on 1 lineal ft. of footing by the allowable unit bearing value of the soil, which is, in this case, 1½ T., or 3,000 lb. per sq. ft.

This theoretical width is then 5,801 ÷ 3,000 = 1.933 ft., say,

practically, 2 ft. The footing course should project from 3 to 4 in. on each side of the foundation wall, so in practice the footing would be made about 30 in. wide, or somewhat larger than assumed in the solution of the problem. Ans.

6. Where the exterior walls of a building are the support for the floors and are of masonry, it is customary, in designing the footings under the columns in the interior of the building, to so proportion them that they will exert a unit pressure on the soil 10 per cent. greater than the footings under the walls exert. This is done so that the greater settlement of the columns will compensate for the settlement that will take place in the body of the masonry owing to the shrinkage of the mortar joints, though this condition does not exist when cement mortar is used.

In order to illustrate these principles, the following solution to part of the example in Art. 3 is given.

The load transmitted through the columns to the footings underneath the column piers includes the live and dead loads, the live load amounting to 80 pounds, and the dead load to 60 pounds, per square foot of floor area. The floor area supported by each column, according to the dimensions given in Fig. 1, is 15 × 20 = 300 square feet; the total floor area from the ten floors amounts to 3,000 square feet. The roof load supported by the columns may be taken as 300 × 25 = 7,500 pounds. The entire load on the footing, in pounds, will then be as follows:

Dead load, 3,000 × 60	= 180000
50 per cent. of the live load, 3,000 × 80 × .50	= 120000
Roof load	= 7500
Total	= 307500

It is advisable to increase the unit pressure on the soil under the column footings above that under the footings of the walls. Since the adopted unit pressure on the soil under the wall is 4 tons, the pressure on the soil under the pier footings should equal 10 per cent. more than 4 tons, or 4.4 tons, and as the load on these footings is equal to 307,500 pounds, or practically 153.75 tons, the area of the

footings under the columns will equal $153.75 \div 4.4 = 34.94$ square feet, so that if the footings are square the dimension of the side will equal the square root of 34.94, or 5.91 feet.

EXAMPLES FOR PRACTICE

1. The load transmitted by a cast-iron column to a foundation pier is 200,000 pounds. What should be the size of the base of the pier, provided that the soil is compact gravel and sand, with a safe bearing capacity of 6 tons per square foot? Ans. 4.08 ft. square

2. A brick pier rests on stiff blue clay having a safe bearing value of 2.5 tons per square foot. If the footing is 5 feet square, what load will it safely sustain? Ans. 125,000 lb.

3. A cast-iron column in a theater supports 400 square feet of the first floor and 200 square feet in each of the two galleries. The dead load on the column is 25 tons. The floor supports a live load of 120 pounds per square foot. What will be the size of the square concrete footing under the column pier according to conservative practice, provided that the soil will safely support $2\frac{1}{2}$ tons per square foot? In this problem the 10 per cent. increase for interior columns need not be figured, since the exterior walls are of the skeleton construction in which the entire weight is carried by wall columns. Ans. 4 ft. 11 in.

4. Determine the size of the square footing required for the cast-iron column subject to the conditions of loading shown in Fig. 4. The lower floor of the building is used for storage, while the upper floors are used for light manufacturing purposes. The amounts of the loads and the bearing value of the soil necessary for the solution of the problem are given in the figure. In calculating the dimensions of the footings, the usual percentages of the live loads are to be considered, and the conservative percentage of deduction may be made. Ans. 4 ft. 5 in.

FOUNDATION PIERS

7. The **foundation piers** under columns should be carefully designed, for usually the support of all the floors above depends on the foundation piers, and any settlement or failure in these footings is likely to cause settlement in the floors or the collapse of the building. Too much care cannot be taken to ascertain that the foundation under such piers is secure. A notable instance of neglect in this matter occurred in New York, where a column pier was built over

an old cistern. The caving in of the cistern, after the building had been completed, caused the column to drop several feet and practically destroyed the building.

Column piers are usually of brick and are built on concrete or stone footings, the brickwork in turn supporting a capstone, which, in buildings of the ordinary class, is usually of sandstone, preferably of bluestone, though sometimes of granite or even limestone.

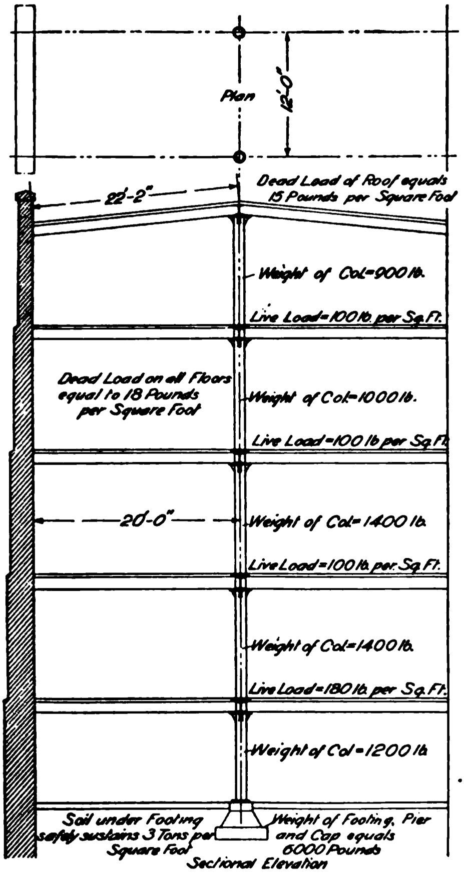
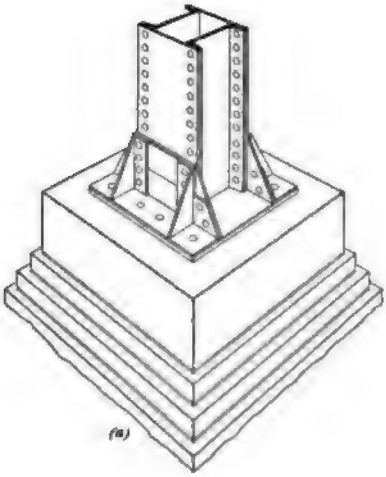


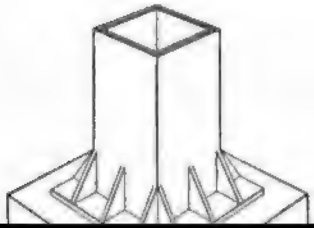
Fig. 4

8. Where the load on the column is great and the capstone is large in area, it is necessary to place a cast-iron base beneath the column. This base distributes the pressure over a greater surface of the capstone and is a great convenience in erection. The particular advantage gained by using the base is the fact that after the piers have been built, the bases can be carefully set, leveled, and lined, and offer an immediate means for locating the columns. On this account the separate cast-iron base is often used for light construction, and such a base supporting a cast-iron column is shown in Fig. 2 (*b*). Commonly, no cast-iron

base is used, in which case the column rests directly on the capstone, as shown in Fig. 5 (*a*) and (*b*). View (*a*) shows the capstone under the base of a structural steel column, while view (*b*) shows a cast-iron column resting directly on the capstone.



(*a*)



9. The design for a heavy foundation pier is shown in Fig. 6, and the proportions that are given are applicable to all column piers. The footing, if of concrete, should never be less than 12 inches and is sometimes made 16 or 18 inches in thickness. The projection of the footing beyond the brickwork should not be greater than one-half the thickness of the footing and never more than 8 inches. The batter of the brickwork at the sides should not be greater [redacted]

have such a height that the edge of the web *d* makes an angle of at least 60° with the base, and wherever possible, the base webs should be provided directly under the principal webs of the column. This cast-iron base plate should be so designed that the greatest compressive stress per square inch of section shall not exceed 12,000 pounds. The base plate should be **faced**, that is, finished by planing or milling on both the top and bottom surfaces, and in setting it, it may be placed directly on the stone cap, if the top surface of the stone is axed level and true; if not, it should be set on a thin bed of cement mortar, or it may be shimmed up to a level with small iron wedges *a*, *a*, Fig. 7, and grouted, the grouting being performed by damming the outside edge of the casting, as shown, and pouring a semifluid paste of cement through holes in the bottom flange of the casting or in the hollow of the casting.

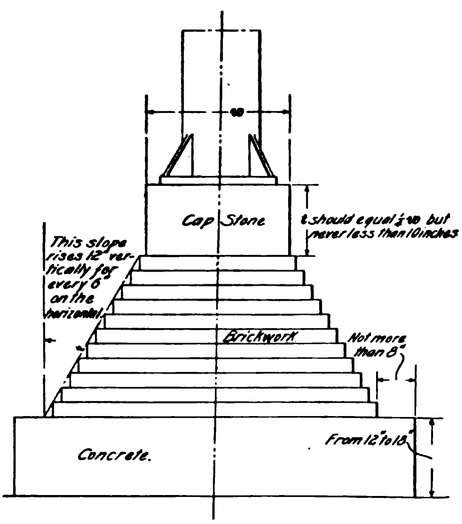


FIG. 6

10. In proportioning the footings under piers, it is first necessary to figure the loads on the column and to obtain the total load on the footing at the surface of contact with the soil; the area of the footing is then proportioned by dividing the total load by the safe unit bearing value of the soil. It is then necessary to determine the size of the capstone, which is done by first determining its area at *b b*, Fig. 2 (*b*), by dividing the total pressure on the surface *b b* due to the load on the column by the safe unit bearing value

of the brickwork, which is given in *Materials of Structural Engineering*, Part 2. If the capstone is square and the area at *b b* is known, the dimensions of its sides may be found by taking the square root of the area. If for any reason a rectangular capstone is necessary, one side should be assumed and the other found by dividing the area required at *b b* by the assumed side. Having the size of the footing and the size of the capstone, and knowing that the offset of the footing course should not be greater than 8 inches and also that the

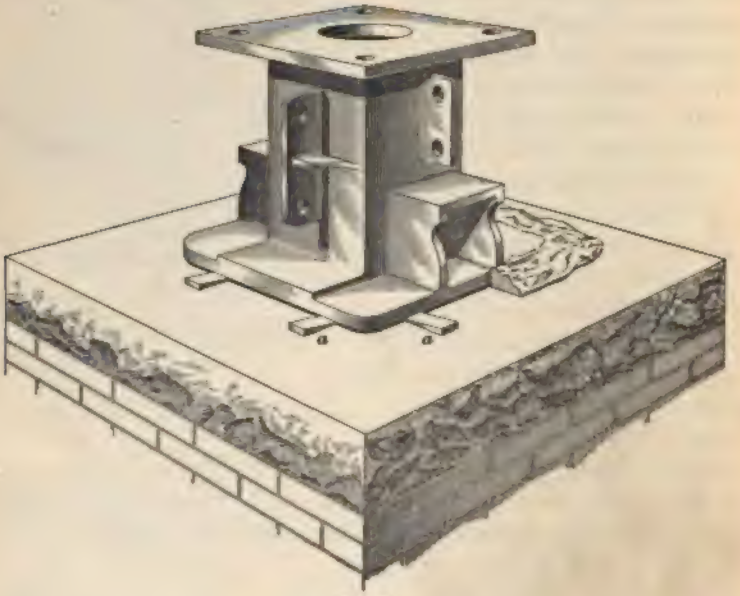


FIG. 7

batter of the brickwork should not be greater than 6 inches to 1 foot in height, the height from the level *e* to *f*, Fig. 8, may readily be determined by drawing upwards a line from the point *e* at the allowable slope until the point *f* is obtained at a distance from the center line equal to one-half the length of the side of the capstone. The first course of brickwork should project one-half the thickness of a course beyond the edge of the capstone. From this it will be seen that

the height of the pier is fixed by the dimensions of the footing and the capstone and by the allowable batter of the brickwork. If the height determined by these conditions cannot, for some peculiar reason of locality or construction, be adopted, the capstone must be made larger, so that the height of the footing can be reduced, or else spread or grillage footings must be used, as explained in *Heavy Foundations*.

11. The length of the sides of the cast-iron base is determined by the allowable bearing value of the capstone; that is, the dimensions of the base are determined when the base is square by dividing the total load on the column by the allowable unit bearing value for the stone, and by extracting the square root of the quotient. When the base is rectangular, one side is assumed, as was explained in connection with the determination of the size of the capstone. The unit bearing values for structural building stone are given in *Materials of Structural Engineering*, Part 2.

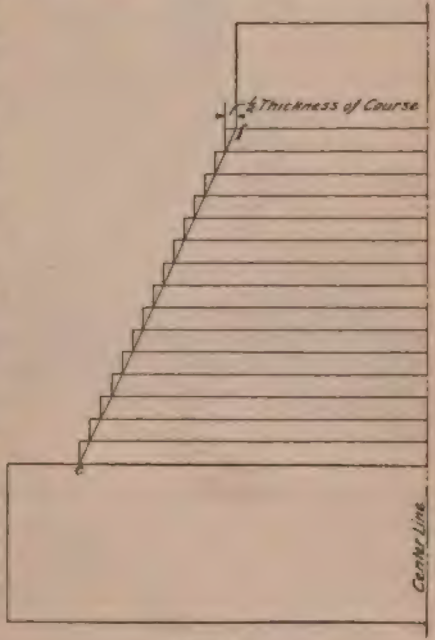


FIG. 8

12. The application of the method of designing a foundation pier for a column is shown by the following example:

The load on a cast-iron base supporting a wood column is 200,000 pounds. Design, for this column, a foundation pier made of brickwork in cement mortar with concrete base and granite cap. The soil under the pier, being compact gravel and sand, can safely sustain 6 tons per square foot. The load

on the soil, transmitted through the pier, is 200,000 pounds, or 100 tons. Assuming a maximum load on the soil of only one-half of its safe bearing value, we have 100 ÷ 3 = 33⅓ square feet that the base of the concrete must cover. The area of a square 5 feet 8 inches on a side is 32.11 square feet. Therefore, the concrete base should be about 5 feet 9 inches square. Next, it is required to find how large the brick pier should be at the top, or at the surface *a*, Fig. 9. The bearing value of brickwork in Portland cement mortar is assumed to be 200 pounds per square inch. The load coming on the pier is 200,000 pounds; therefore, the required area at the surface *a* is 200,000 ÷ 200 = 1,000 square inches; or, 1,000 ÷ 144 = 6.94 square feet. The area of a square 2 feet 8 inches on a side, being 7.11 square feet, the brickwork at the surface *a* should be 2 feet 8 inches.

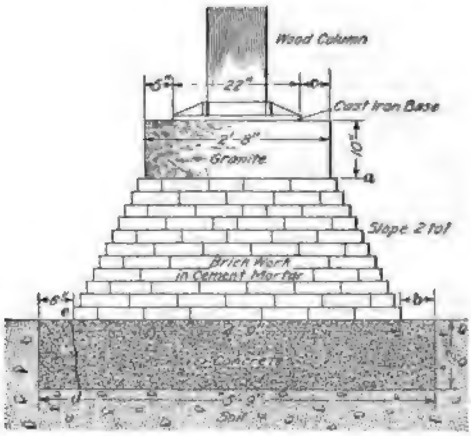


Fig. 9

It is now required to determine the area of the brickwork at the bottom, or where it rests on the concrete base. The concrete base will, with safety, support 150 pounds per square inch. As the pressure on it is 200,000 pounds, the area, in square inches, at this point must be 200,000 ÷ 150 = 1,333 square inches, or 9.25 square feet. A square whose sides are 3 feet 1 inch, has an area of 9.5 square feet, which would, in theory, be the area of the brick pier next to the concrete. In the case under discussion, represented by Fig. 9, it happens, however, that the area of the concrete base required is 5 feet 9 inches on a side, while the greatest limit to which it may extend beyond the brick pier is,

according to good practice, about 8 inches, due to its liability to break off at the line *de*; the adoption of the theoretical area of the pier at this point is, therefore, inconsistent with good practice, the edge of the brick pier being carried out to within 6 inches of the edge of the concrete, regardless of dimensions obtained in calculating the required area for brick piers bearing on the concrete bases. As the granite cap bears on the brickwork at the surface *a*, its area is governed by the bearing strength of the brickwork, and is required to be, as previously found, 2 feet 8 inches square. The area of the cast-iron base is governed by the permissible unit pressure on the granite capstone, which is 700 pounds per square inch. Therefore, $200,000 \div 700 = 285$ square inches required to be covered, which means a cast-iron base about 17 inches square. The distance that the capstone extends beyond the base should not be over one-half of the thickness of the capstone. The capstone being in thickness one-fourth the width of the side, its thickness in this case would be one-fourth of 2 feet 8 inches = 8 inches; but, as it should never be less than 10 inches, this thickness is assumed. The distance *c* in this case is, then, with safety, placed at one-half of 10, or 5 inches. The cast-iron base would then necessarily be 22 inches square.

Fig. 9 shows this pier foundation drawn to scale, according to the figures reached by the preceding calculation, in which the weight of the pier was not considered, such exactitude not being generally required in ordinary building construction.

13. For piers with vertical sides, as illustrated in Fig. 10, the bearing strength is calculated in the same manner as for those with battered sides. Such piers, of necessity, supporting concentrated loads, should be built with special care. It has been stated in connection with single stones used as columns that there is very little reliable data from which their strength may be calculated. The same is true of tall piers of brickwork or stone masonry. A good practical rule for rough-stone piers is that their height shall not exceed four times the least width; for brick piers, the height should not exceed six times the least width, while carefully

constructed cut-stone piers may be so proportioned that their height is not greater than eight times the least width, the joints of the latter being not greater than $\frac{3}{8}$ inch. Within

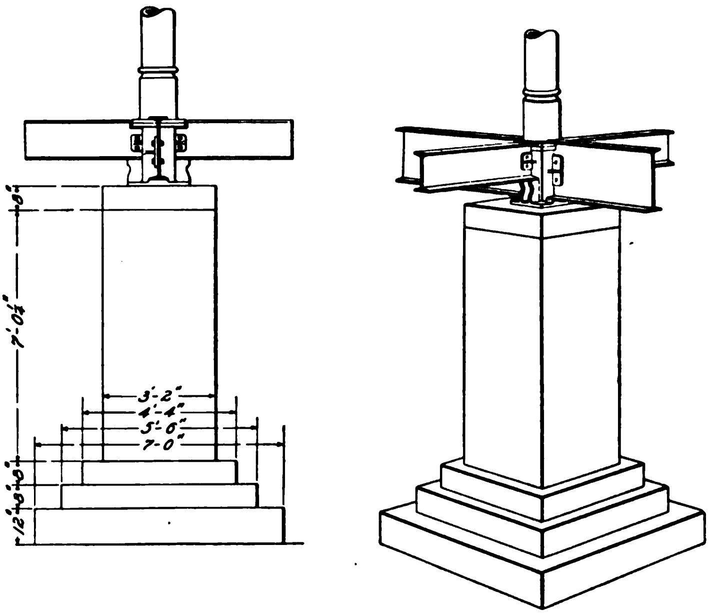


FIG. 10

these proportions, the allowable bearing value of the masonry may be assumed in calculating the safe strength of the piers. All piers of importance should be set in good cement mortar, preferably Portland.

EXAMPLES FOR PRACTICE

1. Determine the size of a brick pier at the top and at the base. The pier is to be square in plan and it is required to support a load of 150 tons. The brickwork is laid in Portland cement mortar and has a safe bearing value of 250 pounds per square inch. The concrete used for the footing will sustain, safely, a load of 100 pounds per square inch.

Ans. { At top, 2 ft. 11 in.; At base, 4 ft. 7 in.

2. What will be the minimum height of the brick pier in example 1, provided that the slope recommended in good practice is not exceeded?

Ans. 20 in.

3. What would be the cubical contents of a concrete footing to support a load of 200 tons on a clay and sand foundation, the safe bearing value of which is 5 tons per square foot. The minimum thickness of the concrete footing may be considered, as its projection is slight. Ans. 40 cu. ft.

FOUNDATION WALLS

14. Foundation walls include all walls and piers built below the curb level for the support of walls, piers, columns, girders, posts, or beams. Such walls are usually proportioned after the wall of the building has been determined on. In proportioning the walls of a building, it is not only the bearing strength of the wall that is to be considered but also the proportion of its thickness to its height. Certain proportions have been found best by experience, and are usually prescribed by the building laws of the particular city in which the building is to be erected. Reliance should therefore rather be placed on the rules and data of experience and the building laws followed than on theoretical analysis. It is assumed that the thickness of the building wall where it rests on the foundation has been determined and the foundation remains to be proportioned.

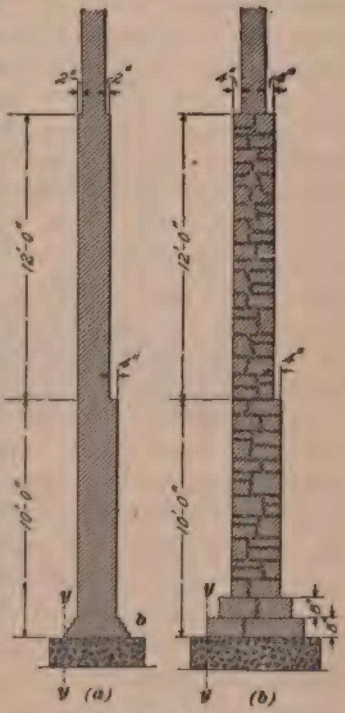


FIG. 11

Foundation walls are of brick, stone, or concrete. Where they are of brick, as in Fig. 11 (*a*), the upper 12 feet should be at least 4 inches thicker than the wall that it supports and the thickness should be increased 4 inches for each additional 10 feet in depth; where the

footing is stepped, as shown at *b*, the projection should not exceed $1\frac{1}{2}$ inches for each brick in thickness.

If the foundation wall is of stone, as shown in Fig. 11 (*b*), the upper 12 feet of the foundation wall should be 8 inches thicker than the building wall and the thickness should be increased 4 inches for the lower 10 feet. The stones at the foot of the wall should be at least 2 by 3 feet and not less than 8 inches thick. For walls 24 inches thick and under, bond stones extending clear through the wall should be placed every 3 feet in height and 3 feet in length. Walls over this thickness should be bonded with stones at least 2 feet long, the two stones from the opposite faces of the wall overlapping. Such bond stones should be provided every 6 square feet of wall surface. All headers in foundation walls should be at least 12 inches wide and 8 inches thick, and none of the stones should be laid other than on their natural beds. The stonework is preferably laid up in cement mortar, all of the spaces and joints being thoroughly filled. The footings under either brick or stone walls, as shown in Fig. 11 (*a*) and (*b*), should be so proportioned that there is no danger of their failing by transverse stress along the line $y y$, and the projection of the footing should never exceed one-half the thickness.

Concrete footing courses under foundation walls should never be less than 12 inches thick and they should always project at least 12 inches beyond the foundation wall, except where the bottom of the wall is not stepped, in which case the projection of the concrete beyond the wall may be reduced to 6 inches. While the general practice is to make the projection of the concrete beyond the lower course of the foundation wall equal to one-half the thickness of the footing, conservative practice usually limits this projection to 6 or 8 inches, even though the thickness is greater than 12 inches.

15. In cities where the building sites are limited in number and area and ground becomes of great value, the buildings are built close together, in order that each square foot of surface may be utilized, the back of one foundation

wall being in contact with the wall of the adjacent building. Where the foundations are carried to the same depth, no difficulty arises from such practice, but where an old building is built close to the lot line and a building having deeper foundations is to be put up beside it, precaution must be taken in making the new excavation in order to avoid disturbing the foundations of the old building. Fig. 12 shows the excavation adjacent to the foundations of an old building. As shown, the stability of the earth underneath the footings

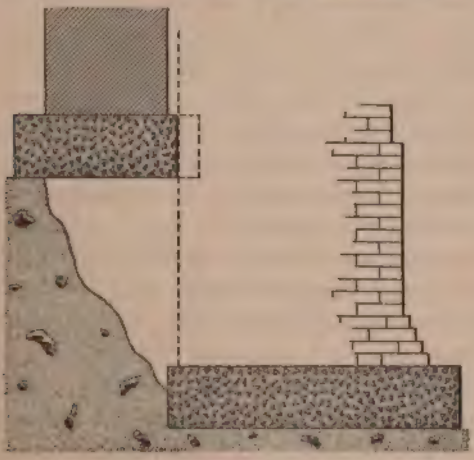


FIG. 12

of the old wall is greatly disturbed, for the soil tends to slide or cave and leave the footing unsupported. The usual method employed to obviate this difficulty and to preserve the foundation and the stability of the old building is to **underpin** the foundation wall; this consists of temporarily supporting the wall and building beneath the present footings, a new foundation wall extending to the depth of the foundations for the new work.

REQUIREMENTS OF THE BUILDING LAWS

16. When the work to be constructed is in a large city, it must comply with the building laws and ordinances of that place. It is from the building laws of the city of New York

that the ordinances of many smaller cities are framed, and as these laws have only recently been revised, that portion relating to foundation walls is given here and is recommended for study, as it offers reliable data for designing.

1. *Foundations*.—Every building, except those erected on solid rock or on wharves and piers on the water front, shall have foundations of brick, stone, iron, steel, or concrete laid not less than 4 feet below the surface of the earth, on the solid ground or level surface of rock, or on piles or ranging timbers when solid earth or rock is not found.

2. *Foundation Walls*.—These walls shall be built of stone, brick, Portland cement concrete, iron, or steel.

If built of rubble stone, or Portland cement concrete, they shall be at least 8 inches thicker than the wall next above them to a depth of 12 feet below the curb level; and for every additional 10 feet, or part thereof, deeper, they shall be increased 4 inches in thickness.

If built of brick, they shall be at least 4 inches thicker than the wall next above them to a depth of 12 feet below the curb level; and for every additional 10 feet, or part thereof, deeper, they shall be increased 4 inches in thickness.

3. *Base Course*.—The footing, or **base course**, shall be of stone or concrete, or both, or of concrete and stepped-up brickwork, of sufficient thickness and area to safely bear the weight to be inposed thereon.

If the footing, or base course, be of concrete, the concrete shall not be less than 12 inches thick.

If of stone, the stones shall not be less than 2 by 3 feet, and at least 8 inches in thickness for walls; and not less than 10 inches in thickness if under piers, columns, or posts.

The footing, or base course, whether formed of concrete or stone, shall be at least 12 inches wider than the bottom width of walls, and at least 12 inches wider on all sides than the bottom width of said piers, columns, or posts.

If the superimposed load is such as to cause undue transverse stress on a footing projecting 12 inches, the thickness of such footing is to be increased so as to carry the load with safety.

For small structures and for small piers sustaining light loads, the Commissioner of Buildings having jurisdiction may, in his discretion, allow a reduction in the thickness and projection for footings or base courses herein specified.

All base stones shall be well-bedded and laid crosswise, edge to edge.

4. *Stepped-Up Footings.*—If stepped-up footings of brick are used in place of stone, above the concrete, the offsets, if laid in single courses, shall each not exceed $1\frac{1}{2}$ inches, or if laid in double courses, then each shall not exceed 3 inches, setting the first course of brickwork back one-half the thickness of the concrete base, so as to properly distribute the load to be imposed thereon.

5. *Inverted Arches.*—If, in place of a continuous foundation wall, isolated piers are to be built to support the superstructure, where the nature of the ground and the character of the building make it necessary, in the opinion of the Commissioner of Buildings having jurisdiction, inverted arches resting on a proper bed of concrete, both designed to transmit with safety the superimposed loads, shall be turned between the piers. The thrust of the outer piers shall be taken up by suitable wrought-iron or steel rods and plates.

6. *Grillage in Foundations.*—Grillage beams of wrought iron or steel resting on a proper concrete bed may be used. Such beams must be provided with separators and bolts and be enclosed and filled solid between with concrete, and be of such sizes and so arranged as to transmit with safety the superimposed loads.

7. *Headers in Stone Walls.*—All stone walls 24 inches or less in thickness shall have at least one header extending through the wall every 3 feet in height from the bottom of the wall, and every 3 feet in length, and if over 24 inches in thickness, shall have one header for every 6 superficial feet on both sides of the wall, laid one on top of the other to bond together, and running into the wall at least 2 feet.

All headers shall be at least 12 inches in width and 8 inches in thickness, and shall consist of good flat stones. No stone shall be laid in such walls in any other position than on its

natural bed, and none shall be used that does not bond or extend into the wall at least 6 inches. The stones shall, in all cases, be firmly bedded in cement mortar and all spaces and joints shall be thoroughly filled.

COINCIDENCE OF CENTER OF GRAVITY AND CENTER OF BEARING

17. In designing foundation footings it is of importance that the line passing through the center of gravity of the wall section is coincident with the center of the bearing area of the footing. This is explained by reference to Fig. 13 (*a*), which shows a high wall whose thickness has been reduced at the several stories so that the center of gravity of the wall is well to the outside, as shown by the line *y y*; therefore, in order that this line may coincide with the upward pressure acting at the center of bearing as at *a*, the footing *b* must project as little as possible, while the footing at *c* should be stepped or projected sufficiently to draw the center of the bearing area over until it coincides with the line *y y*. The footing properly designed is shown in Fig. 13 (*b*).

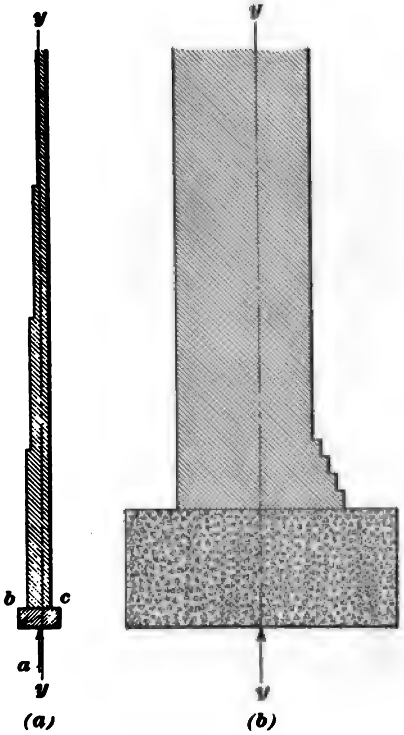


Fig. 13

If the center of the bearing area of any wall is within the point at which the vertical line passing through the center of gravity of the wall intersects the base, the wall will tend to bulge outwards, while if the center of the bearing area

is without this line, the wall will tend to fall or bulge inwards. These two conditions are shown in Fig. 14 (*a*) and (*b*), respectively. In each case the center of application of the upward pressure from the soil, or the center of the bearing area, is represented by *P*, while the weight of the wall and its point of application is shown by *W*. This important principle in the proportioning of foundations may be stated concisely by the following law:

Law.—*The center of the bearing area of all footings should, wherever possible, coincide with the point of intersection made by a vertical line passing through the center of gravity of the load and the bottom line of the footing; and in designing the footings, rather than there being any danger of the center of the bearing area falling inside of the point of application or center of gravity of the weight of the wall, the footing should be purposely designed so that the center of the bearing area will fall without the center of gravity of the weight.*

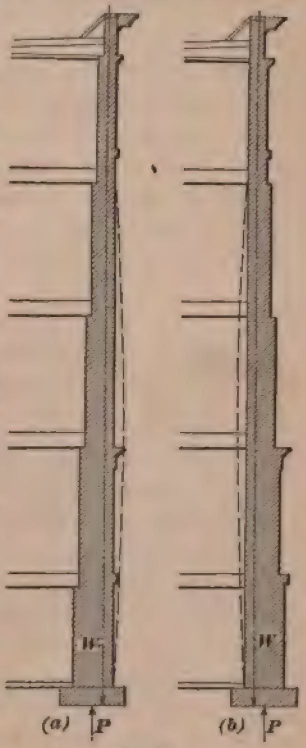


FIG. 14

The observance of the latter portion of this law is advisable from the fact that, while a wall may be thrown outwards, it cannot readily be forced inwards. The only resistance of a wall to outward bulging or leaning, outside of its own rigidity, is the bonding into the cross-walls by masonry or the tying-in of the wall by beam anchors, while from inward bulging or leaning, it is almost irresistibly reenforced by the masonry partition walls and by the lateral rigidity of the floor construction.

18. Carelessness in the design of footings does not necessarily cause the collapse of the structure, but it will

almost invariably cause unequal settlement in the building and produce unsightly cracks in the walls. The mistake is not alone made in proportioning the footings with too small an area, for frequently the footings are of uniform width throughout, and are designed with a much greater area at one point than is required, while in another place the area is just sufficient to safely sustain the load. In consequence, the wall or pier supporting the lighter load and having a larger area in proportion has very little settlement, while another portion of the building, or even the same wall, more heavily loaded, settles considerably, and causes cracks in the walls or the dropping of the arches. Here the trouble is caused, not by an insufficient area of footing, but from the fact that certain portions of the footings have an area of too great an extent for the load on them.

DEFECTIVE DESIGN OF FOUNDATION WALLS AND FOOTINGS

19. The defective design of foundation footings is generally the cause of the failure of foundation walls to maintain the superimposed work in alinement. Unsightly fractures in the upper walls of a structure may usually be traced to poorly proportioned footings. As stated in the preceding article, the defect is not always due to the fact that the foundation footings have not sufficient bearing area, for frequently the cause of defective foundations is found in the fact that the footings have too wide an area, and that in consequence the distribution of the pressure does not give a center of effort which coincides with the center of gravity of the superimposed weight.

With the advent of skeleton construction, in which not only the floor loads but also the weights of the walls are carried on steel columns, the system of foundation construction known as **isolated piers**, has been introduced. The advantage of this design is that each foundation footing can be accurately proportioned for the load that it will be required to sustain. While it is not practical to erect all

structures on isolated piers, yet the principles governing this character of design can be applied to advantage in the design of foundations.

The primary causes of defective foundation design are explained in the following descriptions and illustrations. Though it is sometimes impossible to avoid defective foundation construction, yet by intelligent study of some of the

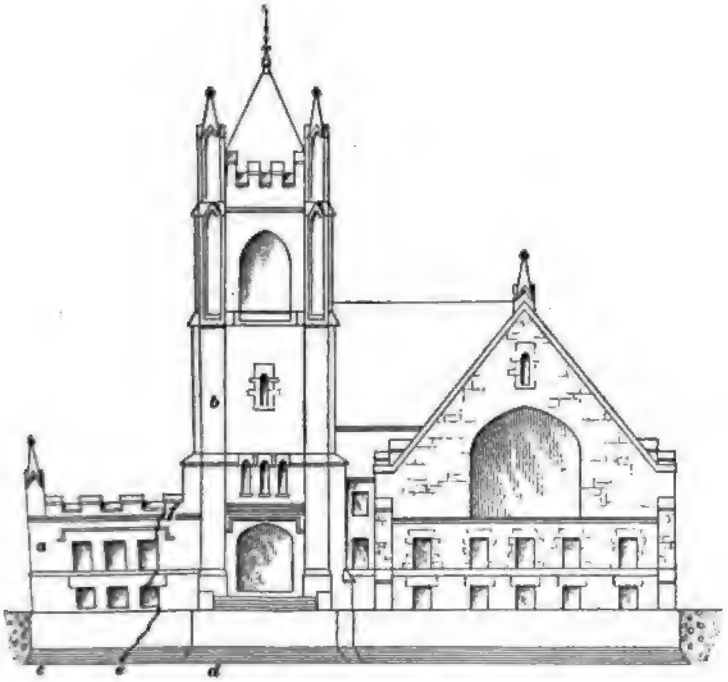


FIG. 15

frequent causes of failure, a foundation design may be greatly improved, and lamentable defects and sometimes calamitous results may be avoided.

20. Disproportion of Foundation to Load. — In Fig. 15 is shown one of the most common defects of construction caused by a poorly designed foundation. It is the effect that is apt to be produced if the same foundation

footings as are used under the heavy portions of the building are extended under the lighter portions. The pavilion, porch, or annex *a* is of inconsiderable weight when compared with the weight of the tower *b*. If, therefore, the same footings are used under this portion of the building as are employed for the support of the tower, and considerable settlement takes place, the tower will settle more rapidly than the light wall enclosing the pavilion *a*. The result will be that the footing *c* will remain at the original level, while the footings *d* carrying the superimposed load of the tower will settle below their original level, and consequently the wall will crack along the line of *least resistance*. In this instance, the line of fracture would probably extend from *e* to *f*.

While the damage done through this defect in foundation planning is not necessarily calamitous, it is certainly desirable to avoid the disfigurement of the building. It may be avoided by accurately proportioning the footing for the load that it is required to sustain and providing footings at *c* proportionately smaller than the footings at *d* for the respective weights that they are required to support.

In constructing such work, it is well, if possible, to build the tower in advance of the other work, so that the weight on the footings under the tower will be realized at the same time that the weight on the footings under the balance of the building has been attained. Ordinarily, the portion of the building at *a* would be completed before more than one-third of the tower had been built, so that the footings at *c* would sustain their full load, while the footings at *d* would carry only one-third of the superimposed load; consequently, the better practice is to add material during the construction of the tower in the same proportion as used in the construction of the lower portions of the building.

In some instances, it is well to provide a slip joint between a heavy tower and the attached building. In this way, the additional settlement of the tower is provided for in the slip joint, and no unsightly lines of fracture are developed through the walls.

21. Extension of Footings Beneath Area of Building.—Owing to the unstable nature of the soil, concrete footings have been extended beneath the entire building. These footings, which are usually made of concrete from 2 to 6 feet in depth, form a raft on which the building is practically floated. In several instances, these footings have been found entirely defective and have produced results that finally led to the demolition of the structure.

The defect in this system is explained more readily by reference to Fig. 16. In view (*a*) is shown a brick wall supported on the ordinary footing *a b c d*. When so constructed, even though the footing may be of great width, the

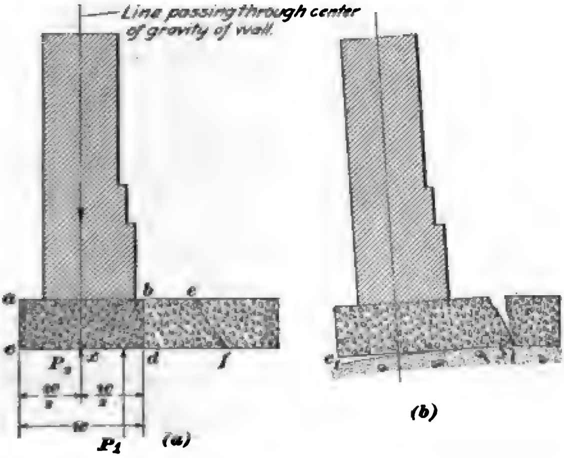


FIG. 16

center of pressure of P_1 is applied at the point x, which is located in the center of the width of the footing cd. When carefully designed, the center of pressure P_2 should coincide with the line passing through the center of gravity of the wall, which would be the condition required in a well-designed footing. Assume that, instead of the wall resting on a carefully proportioned footing, as at *a b c d*, the wall of the building is supported on a concrete slab extending beneath the entire building or the portion enclosed by the wall. The concrete, when of greater depth than 1 or 2 feet,

has considerable transverse resistance, so that when settlement occurs the portion supporting the wall will break away from that portion of the slab that sustains no weight, the fracture probably being along such a line as ef. The footing therefore, instead of having the width cd, will be as wide as the distance from c to f and the center of pressure will be moved to P_1. The location of P_1 is far within the line passing through the center of gravity of the wall, so that the wall will be thrown outwards, as in view (b), this tendency being greatly increased by the additional bearing created at the edge c_1.

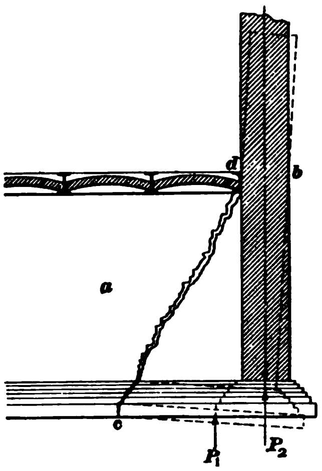


FIG. 17

The precaution suggested in this explanation is to eliminate from the foundation of a structure all concrete slabs of considerable area that have not sufficient transverse resistance to allow the entire load on them to be symmetrically placed and carried with reference to the center of pressure. There are instances where a heavy tower can best be carried on a concrete footing extending entirely under the area covered by the tower. When this construction is employed, the concrete must be of such a depth, or must be reenforced in such a manner by the introduction of steel bars or rods, that no fracture will develop throughout its surface and the single slab will act as a unit in sustaining the weight of the tower.

22. Defective Foundations Caused by Attached Cross-Walls.—In Fig. 17 is shown another means by which the center of the bearing area for a wall, or the center of pressure, is sometimes disturbed from its correct position beneath the center of gravity of the wall. The partition wall a is provided with a heavy footing course that has little

superimposed load; besides, it is securely built and bonded into the main wall *b*. There are several of these partition walls throughout the length of the building, and from the fact that the main wall is heavily loaded and considerable settlement takes place, the cross-walls or partition walls are fractured as shown by *c d*. In this manner, a considerable area is added to the footings of the main wall, causing the center of pressure to move from P_2 to P_1, the tendency

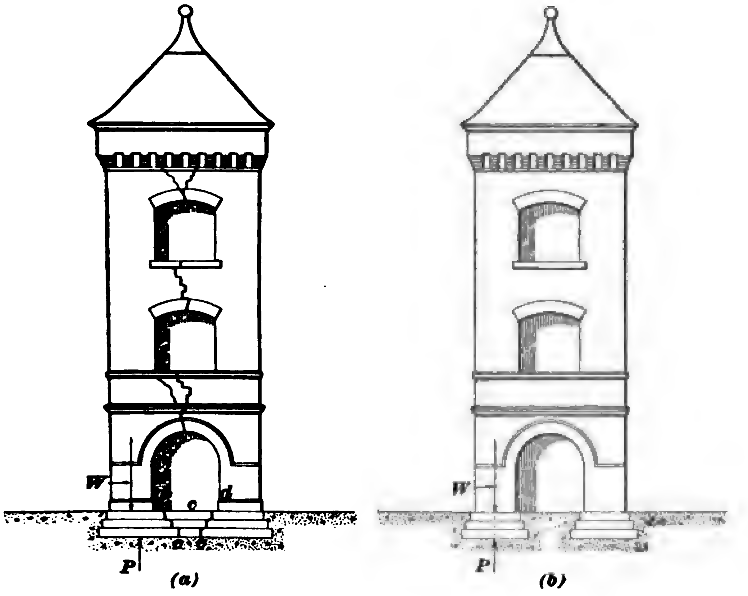


FIG. 18

being to bulge or throw the wall outwards. Where such a condition is likely to exist, either the footings of the cross-walls must be carefully proportioned and reduced to the minimum width, or a slip joint must be provided between the cross-walls and the main wall.

23. Extension of Footings Under Large Openings.—Two methods of constructing the footings under a tower having a large doorway at the ground level are illustrated in Fig. 18. From an inspection of the footing in view

(*a*), it is evident that there is no load on the portion of the footing *c* and that should a fracture occur in the footing this portion will remain in place, breaking away from the footing beneath the pier along the lines *de*. In this manner, the area of the footing underneath the pier or corner of the tower is extended from the corner and toward the opening so that the center of pressure is moved inside the center of

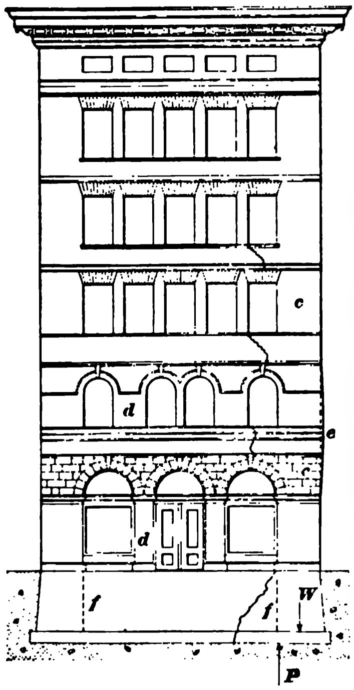


FIG. 19

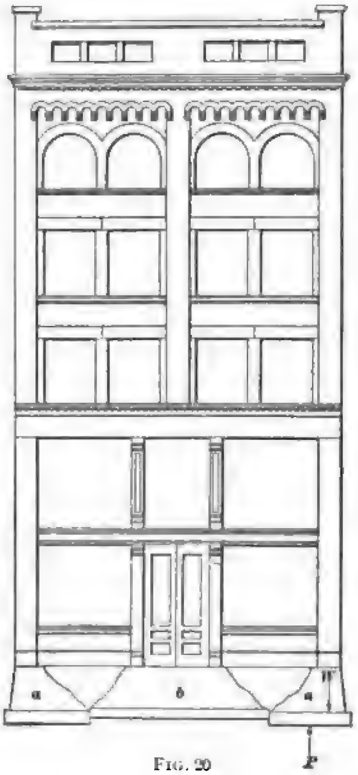


FIG. 20

gravity of the weight. This tends to throw the corners of the tower from each other, spreading the tower and fracturing the arches, sills, and band courses, as shown in the illustration. With the footings properly proportioned for the superimposed load, as in view (*b*), the center of pressure is caused to coincide with the center of gravity of

the weight, and the piers of the tower will settle vertically with no tendency to spread.

A similar condition is shown in Fig. 19. Here the corner *c* of the building is heavily loaded, while the central portion of the front of the building *d d* is of light construction. The corner mass or pier *c*, owing to its greater weight compared with the remaining portion of the front, settles more, producing the fracture shown and causing a spreading or bulging at *e*. To avoid this condition, it would not be bad practice to lighten the footings between the lines *f*, *f*, or to carry all of the central portion of the basement story and the central portion of the remaining stories on heavy girders at each floor.

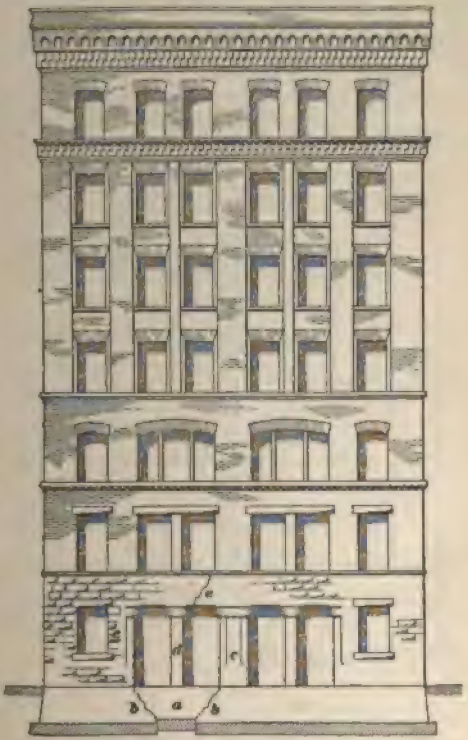


FIG. 21

This defective foundation construction is again illustrated in Fig. 20, and though the architectural treatment of the building is poor, such a design is quite common. The same remarks apply to this figure as were made in reference to Fig. 19.

24. Defects in Foundations Producing Broken Mullions and Band Courses.—In Fig. 21 is shown a façade that is subjected to damage by defective foundation construction. Owing to the light load on the foundation *a*,

fractures are likely to occur at *b*, *b*, the portion of the foundation marked *a* remaining in place while the adjacent portions settle. It may readily be observed, by studying the figure, that if *a* remains in place and the portions *b*, *b* settle, carrying with them the side piers *c*, *c*, the mullion *d* will be crushed, provided that the portion of the masonry *e* remains intact. If the mullion has sufficient resistance to sustain any additional pressure brought to bear on it, the masonry *e* may be fractured as shown.

A condition of this kind is sometimes combatted by the construction shown in Fig. 22, in which the heavier portions of the building, represented by the piers *a*, *a* with their superimposed load, are carried on isolated piers, the weight of the masonry beneath the sills being transmitted to these piers by arches. This construction is particularly applicable to factory work where no cellar or basement is provided and where the building is to be erected on *made ground*, that is, ground that has been filled in. In such instances, the footings of the piers can be laid on the natural soil and a sure foundation obtained. There is a minimum amount of excavation required for this construction, which is of much importance when the building is constructed after the ground has been filled to a new level.

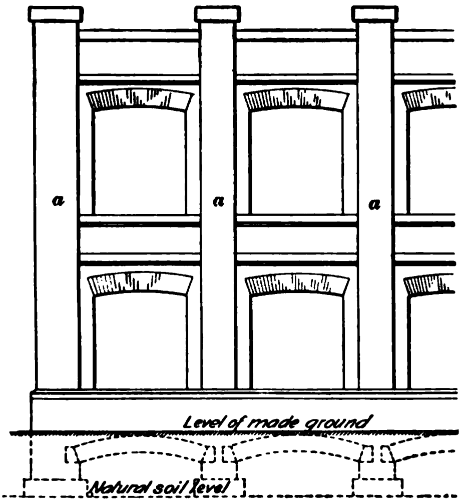


FIG. 22

25. Heavily Loaded Corners.—In Fig. 23 is shown the elevation of a tower with heavy returns or corners and

an intermediate surface with numerous window openings. In Fig. 24 (*a*) is shown a plan of the footing supporting the corner of the tower. The center of gravity of the weight acts along the plane *b b* while the center of gravity of the footing area is along the line *a a*. This condition tends to throw the corners of the tower outwards, and this tendency is greatly aggravated by the fact that the exterior angle of the footing, owing to its shape, cuts into the soil more rapidly than the parallel sides of the footings. Where the soil is uncertain and the appearance of the structure is a consideration, buttresses should be built from the corner of the foundation, as shown in Fig. 24 (*b*). In this way the center of pressure is thrown outside of the center of gravity and the tendency for the corner to penetrate the soil is eliminated. The buttress sets in such a way as to tend to throw the corners of the building together, thus consolidating the mass in the tower.

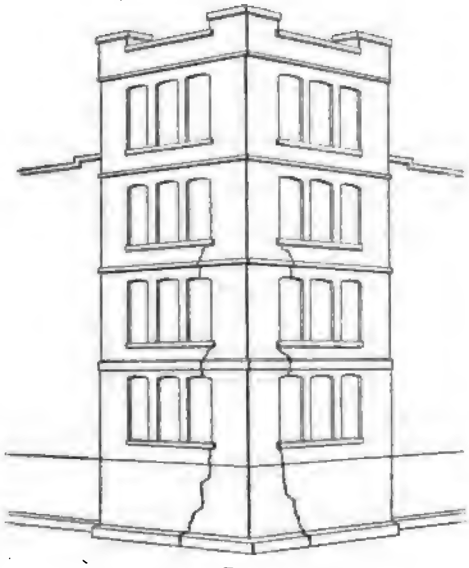


FIG. 23

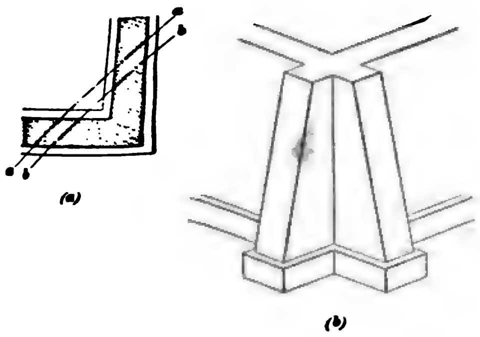


FIG. 24

WALLS

DIVISION OR PARTITION WALLS

26. Division Walls.—By a division wall is understood any wall dividing spaces and having no other weight than its own to support. Walls of this character are acted on by lateral and accidental forces, the principal of which is that due to wind pressure, if the wall is an exterior one.

Interior division walls are called **partitions**, and as such are used to form rooms and corridors; these walls are built between floors.

27. Partition Walls.—As a rule, screens and partitions need not be proportioned to resist any lateral thrust, though they must have considerable resistance, from the fact that they may be subjected to a blow from a falling object.

In a fireproof building, the screens and partitions should be of such thickness and stability as to offer a fair amount of security against failure by the expansion of heated air in the room, caused by fire. If they should fail by this means, their principal purpose would be defeated and great damage might result. The thickness of such walls is frequently decided by the appearance required at the door jambs, if such openings exist. A minimum thickness for brick or terra-cotta partitions not provided with metal stiffeners is one twenty-fifth of their height. Where partition walls extend from ceiling to floor, they may be of any length independent of their height, but screens should not be more than twice their height in length without having intermediate braces; the ends of such screens should always be braced. The top course of a screen should be protected by a coping in long lengths that is anchored down through several courses if there is any danger of its being displaced by knocks.

28. Garden Walls.—These should be built with good strong mortar and, if in particularly exposed places, they should be laid up in the best cement mortar. In masonry work, where the failure of the structure involves serious consequences, a large factor of safety should be used and the adhesive strength of the mortar ignored in proportioning the structure. In such work, however, as **garden walls,** a low factor of safety may be used and the adhesive strength of the mortar may be allowed to enter the calculation. The principal element of resistance required in a garden wall, where it is not necessary to resist the pressure of earth filling forming a terrace, is the pressure due to the wind, which tends to overturn the wall.

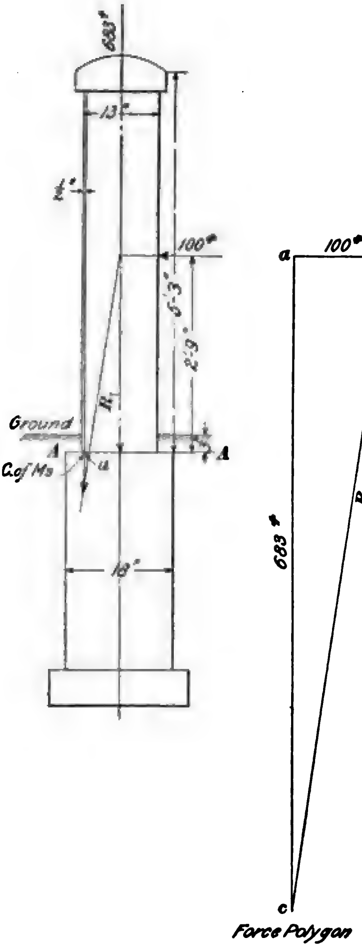


FIG. 25

EXAMPLE.—Consider the action of the wind on the garden wall shown in Fig. 25.

SOLUTION.—It is customary in solving problems of this character to consider only 1 lineal ft. of wall when the wall is straight and without pilasters. If there are pilasters, then the portion of the wall between the centers of the pilasters is considered. The wall subjected to analysis in the present example is straight and without pilasters, so that the stability of a portion of the wall 1 ft. in length will be

discussed. The velocity of the wind near the surface of the earth is considerably reduced by friction, so that the pressure on the wall in this case will be taken at 20 lb. per sq. ft. The wall is 13 in. thick and rests on a stone foundation 18 in. thick. The wall weighs, approximately, 683 lb. per lineal ft. Having these premises, the wall will be analyzed: first, for rotation about its base; second, from crushing at the edge of the wall from the increased pressure caused by the overturning moment of the wind; and third, for shearing resistance in a horizontal plane at the base of the wall.

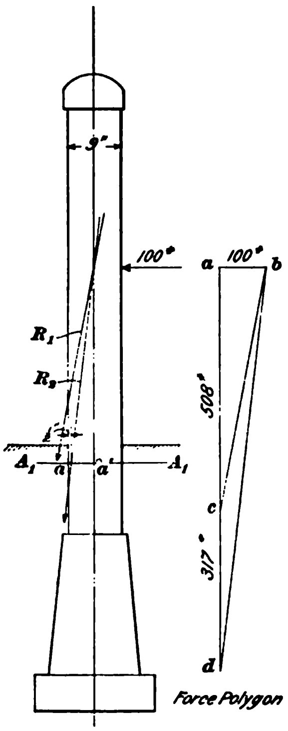


FIG. 26

First.—Since the coping of the wall is curved and the wall extends 3 in. below the ground level, the exposed surface normal, or at right angles, to the wind is 5 ft. high. The wind pressure per lineal foot of wall will be $5 \times 1 \times 20 = 100$ lb. The center of this pressure is applied at the center of the area, which is 2 ft. 9 in. above the top of the foundation wall, or 33 in. If the adhesion of the mortar is not to be considered, that is, if no tension exists at the windward side of the base of the wall, the algebraic sum of the wind moment and the moment of the weight of the wall taken about a center of moments $\frac{1}{2}$ in. from the leeward face of the wall at the point a must equal zero in order to obtain equilibrium. In this instance, the adhesion of the mortar need not be considered, for the moment of the wind is $100 \times 33 = 3{,}300$ in.-lb., while the moment of the wall about the point a is $683 \times 6 = 4{,}098$ in.-lb., from which it is evident that the wall has sufficient stability against overturning to resist the wind pressure, and need not depend on the adhesion of the mortar supplying tension at the windward side of the wall.

If it is desired to lower the cost, the wall can be reduced to 9 in. in thickness, when the adhesion of the mortar must be considered. The moment of the wind on the wall taken on the plane $A A$ remains the same, namely, 3,300 in.-lb., but the moment of the wall about the point a changes, from the fact that both the weight of the wall and its leverage about the center of moments are reduced.

Referring to Fig. 26, which shows a section of the wall reduced to 9 in., the weight of the wall and coping stone per lineal foot is 508 lb.,

and its moment about the point a is $508 \times 4 = 2{,}032$ in.-lb. Thus, the resisting moment of the wall against overturning due to wind pressure is $3{,}300 - 2{,}032 = 1{,}268$ in.-lb. less than is required. This moment must be supplied by the transverse resistance of the wall, say, on the plane $A_1 A_1$. The wall, instead of tending to revolve about the point a' or the center of bearing, rotates about the center of moments a. The formula for section modulus of a rectangular section about an axis coincident with one of its sides, is given in Table I of *Properties of Sections*. From this formula, $S = \frac{b\,d^2}{3}$, so that if the axis is considered $\frac{1}{2}$ in. from the edge of the wall and the section in consequence is taken as 8 in. in depth by 12 in. wide, its section modulus will be $\frac{12 \times 8 \times 8}{3} = 256$.

From *Beams and Girders*, Part 1, $M_1 = S s$, so that $s = \frac{M_1}{S}$. By substitution, $s = \frac{1{,}268}{256}$, or 4.95. The cohesive strength of mortar is very much greater than its adhesive strength, so that the latter value will be considered in the problem. The ultimate adhesive strength of brick or mortar composed of one part Portland cement and two parts of sand, is about 38 lb. per sq. in., so that the tensile resistance is $38 \div 4.95 = 7.6$ times that required. Ans.

Second.—In the analysis of the wall to determine whether its crushing resistance at the extreme edge on the plane $A_1 A_1$ is sufficient, it is necessary to determine the resultant R_1, Fig. 26. The direction and amount of this resultant is determined by laying off, in the force polygon, the line $a\,b$ equal to the wind pressure of 100 lb. and the line $a\,c$ equal to the weight of the wall, or 508 lb., and drawing the line $b\,c$, the latter being the required resultant, when the line of action of the resultant R_1 is drawn from the intersection of the line of action of the wind effort with a line passing through the center of gravity of the wall, the resultant is observed to fall without the edge of the wall. It was ascertained that it would do this, from the fact that tension was required on the plane $A_1 A_1$ in order to prevent overturning. Since, however, this tension has been supplied by constructing the wall of good cement mortar, the action R_2 may be drawn by supplying an equivalent to the tension or adhesion of the mortar in assuming additional weight of wall, as represented by $c\,d$ in the force polygon. This force, R_2, will pass through the center of moments a, and the maximum pressure, which is the weight of the wall per lineal foot, is concentrated at this point. By considering the center of moments as $\frac{1}{2}$ in. in from the leeward face of the wall, the resisting area to crushing may be considered as 1 in. in width and 12 in. long, giving a total bearing area of 12 sq. in. Since the allowable bearing value of good masonry laid in Portland cement mortar is 200 lb. per sq. in., the safe resistance to crushing at the leeward edge of the wall will be

$12 \times 200 = 2{,}400$ lb. The pressure due to the weight of the wall on this total area is only 825 lb., so that the wall is entirely safe from failure by crushing at the leeward edge. Ans.

Third.—From the fact that the stability of the wall is preserved partially by its own weight and partially by the adhesion of the mortar, the wind acting on the wall will cause a horizontal shear at its bed joint and the unit shearing stress produced will be maximum at the plane $A_1 A_1$ rather than on the top of the foundation. To resist this shearing stress there exists the friction between the brickwork and the stone foundation, augmented by the adhesion of the mortar. The frictional resistance is equal to the product of the weight of 1 lineal ft. of wall by the coefficient of friction for brick on stone masonry. The coefficient of friction for these materials is about .74 when the mortar is slightly damp, so that the frictional resistance is $508 \times .74 = 376$. Since the horizontal shear is 100 and the resistance to shear is 376, a factor of safety of 3.76 will exist. This is sufficiently large when it is considered that the adhesion of the mortar also supplies considerable resistance. In this instance, however, the resistance of the mortar bed to the horizontal shear cannot accurately be figured, from the fact that some of its resistance is destroyed by the tensile stress created. Ans.

29. Though the stability of the wall has been determined, the above problem is yet to be completed by analyzing the foundation, which should be laid sufficiently deep in order that the footing may be below the frost line. When the earth is thoroughly compacted on each side of the foundation wall no rotation can possibly occur, and all that need be considered to determine whether there is sufficient bearing area disposed symmetrically adjacent to the center of pressure or resultant force due to the weight of the wall and the wind pressure, is the width of the wall and footing. Frequently, the earth is not as high on one side of a foundation wall as on the other, so that a greater area is exposed to wind pressure on one side and the wall is subjected to a certain lateral thrust created by earth pressure on the other side. When this condition exists, the foundation wall must be designed as a retaining wall and will necessarily extend deeper, in order that the footings may be below frost line on the high side of the wall. In the problem, the ground is assumed as having the same level on both sides of the wall, so that the overturning of the foundation, due to either the wind pressure or earth pressure, need not be considered. It

is necessary, however, to determine at what point the resultant force due to the wind pressure, the weight of the wall, and the weight of the foundation and footing will intersect the base line of the footing course. Referring to Fig. 27 (*b*), the weight of the foundation and footing is laid off vertically from *c*, and the point *d* thus obtained. As may be observed from this diagram, the weight of the foundation wall and footing per lineal foot is 825 pounds. When the point *d* has been determined in this manner and *d* and *b* are connected, both the amount and direction of R_2 are obtained.

This resultant may then be drawn, Fig. 27 (*a*), from the point x, which is the intersection of the line of action of the wind pressure and a line through the center of gravity of the wall. This line will intersect the base line of the footing course at the point x_1, which is at a distance of about $5\frac{1}{2}$ inches from a line passing through the center of gravity of the wall. Since this line of resultant pressure R_2 falls well within the edge of the footing, the foundation may be considered as sufficiently stable and the entire structure will be in equilibrium, if the resistance of the foundation wall and the soil adjacent to the line of action R_2 is sufficient to withstand the vertical component of the resultant pressure at such point. The vertical component on any plane through the foundation, as *b b* or *c c*, due to the combined wind pressure and the weight of the masonry, varies increasingly from the top to the bottom of the wall, from the fact that the weight

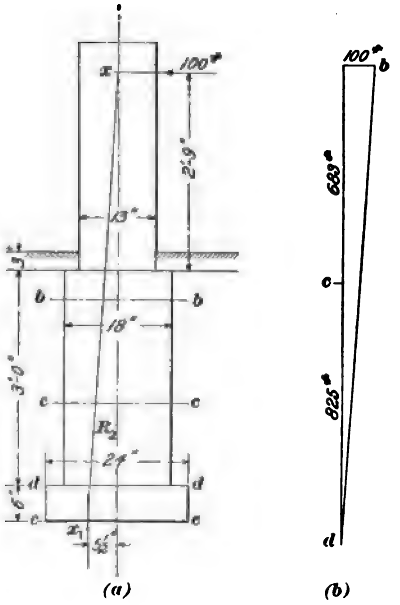


FIG. 27

bearing area of only $2\frac{1}{2}$ inches in width on a foundation soil theoretically approaches a knife edge and would cut into the soil under a far less pressure than its average unit bearing value for an area of reasonable width. Practically, the center of pressure should be at least 3 inches from the edge of the footing. It is only the vertical component of the resultant R_2 that causes direct pressure on the masonry; the horizontal component of this force tends to produce sliding or a shearing action. A review of the conditions, however, reveals the fact that no sliding action can take place below the ground level, so that this may be disregarded. If the wall were built so that the bed joints were at right angles to R_2, it would be theoretically and accurately correct to take the full value of R_2 as the pressure on the masonry.

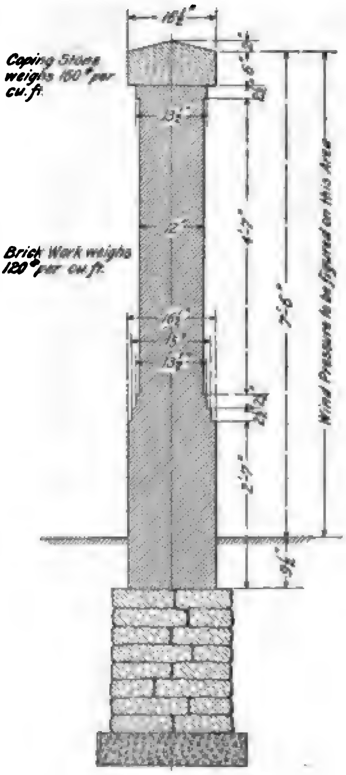


FIG. 29

Fig. 28 shows the wall and foundation designed on a more economical basis, which will be found, by applying the principles explained in this problem, to be correctly proportioned and sufficiently stable to resist any wind pressure that might be created on its exposed surface. It will be noticed from this figure that, while the line of action of the resultant R_2 approaches the edge of the footing, it is still at least 3 inches away. For an important structure supporting great loads, this would be a poor design. It is, however, sufficient for a garden wall if the line of action of the resultant R_2 falls within the edge of the footing by 3 inches.

EXAMPLES FOR PRACTICE

1. Determine the moment of the wind pressure on 1 lineal foot of the garden wall shown in Fig. 29, considering the center of moments ¼ inch from the edge of the wall, adjacent to the foundation. The maximum wind pressure per square foot of surface is to be taken at 20 pounds. Ans. 681 ft.-lb.

2. Calculate the moment of the weight of 1 foot of the wall acting on a line coincident with its axis of symmetry about a center of moments ¼ inch from the edge of the wall adjacent to the foundation. Ans. 9,238 in.-lb.

3. What factor of safety does this wall possess against overturning on the foundation by wind, provided the adhesion of the mortar is neglected? Ans. 1.13

4. Determine, graphically, the point at which the resultant of the wind pressure and the weight of the wall intersects the top of the stone foundation wall. The dimension to determine is the distance of this point from the axis of symmetry or center line of the wall.

SUPPORTING WALLS

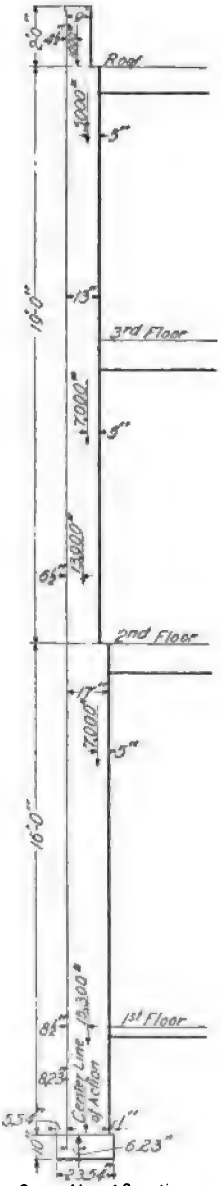
31. The simplest kind of **supporting wall** is one without openings and having a uniform weight for each foot of length, the load being symmetrically placed with respect to the center of bearing of the wall at the base. It is seldom in building construction that such conditions exist, for most supporting walls carry the ends of floorbeams, or joists, on one side and are subjected to lateral wind pressure. From the fact that they are usually built with return pilasters and corbel courses, the center of the weights does not coincide with the center of the bearing.

Supporting walls symmetrically built, uniformly loaded, and not acted on by lateral forces are simple of analysis, the only conditions required being that the compressive stress of the bottom course does not exceed the safe bearing value of the masonry, and that the footings are sufficiently wide to give the allowable safe bearing on the soil.

The loads on walls, especially outside walls of buildings, are generally more or less eccentrically placed with reference to the center of the bearing area, and are frequently concentrated, as at the ends of girders, beams, and trusses. When the concentrated loads on a wall are at close and

uniform intervals, as occurs at the support of floor joists, the wall is regarded as uniformly loaded, for, though sufficient bearing under each joist or beam must be provided, the masonry distributes the loads uniformly a few feet from the top of the wall. Other conditions tending to produce concentrated loads are the openings in walls, for usually openings are placed over each other, so that great loads are often concentrated on small sections of the wall, which become practically piers. In investigating or designing a wall, as in all engineering problems, the weakest section should be ascertained and made sufficiently strong to carry its proportion of the load.

32. The design of a supporting wall is best illustrated by the following example: In Fig. 30 is shown a section through a plain supporting wall of brick. The wall is an exterior one and supports, besides its own weight, two fireproof floors and a fireproof roof, the basement floor being of concrete laid directly on the cellar bottom. Since the steel beams supporting the floors and roofs are placed on centers 6 feet apart, instead of analyzing 1 lineal foot of wall, the portion of the wall included between the centers of two beams will be considered as a unit. The weight imposed on the wall under the end of the steel beams is 7,000 pounds at each floor, while the weight on the wall beneath the end of the roof beams is



Center Line of Reaction or Resistance = 49,200 # Action
FIG. 30

5,000 pounds. The wall consists of the first-floor wall, which is 17 inches thick and 16 feet high, the second- and third-story walls, which are 13 inches thick and have a combined height of 19 feet, and a parapet wall that is 9 inches thick and 2 feet high. The weights of these three walls in round numbers are 15,300, 13,900, and 1,000 pounds, respectively.

The loads from the ends of the floorbeams and roofbeams are considered as acting at the center of their bearings, while the load from each portion of the wall is assumed to act at the center of gravity of that portion. The amounts of these several loads and the positions of their lines of action are shown in the figure. Adding these several weights together, it is found that the total pressure on the footing at the base of the wall is equal to 49,200 pounds.

33. The point of application of this force is, however, to be determined. In order to locate the center of pressure, or the center line of action, shown in Fig. 30, it is necessary to find the center of gravity of all of the loads acting on the footing. In order to determine the position of the center of gravity of all of these loads from the outside edge of the wall at the base, it is necessary to divide the sum of the moments of each of the loads transmitted through the wall around the outside bottom edge of the wall as the center of moments by the sum of the loads or the total amount of pressure at the base of the wall. Referring to Fig. 30, the sum of the moments, in inch-pounds, of the several loads is determined by the following calculation, the tabulation being in sequence from the parapet wall:

Parapet wall	1,000 ×	4.5	=	4500
13-inch wall	13,900 ×	6.5	=	90350
17-inch wall	15,300 ×	8.5	=	130050
Reaction of roof	5,000 ×	8.0	=	40000
Reaction of third-floor beam	7,000 ×	8.0	=	56000
Reaction of second-floor beam	7,000 ×	12.0	=	84000
Sum of moments			=	404900

The total weight transmitted by the wall is equal to 49,200 pounds, so that the distance from the outside face of the wall is $404,900 \div 49,200 = 8.23$ inches.

From this dimension, the center of pressure, or the center line of action, may be located, and is shown on the drawing of the section of the wall as in the figure. If the wall is laid up of good hard brick in lime mortar, its safe allowable bearing value is about 8 tons per square foot, so that as the total load is 49,200 pounds, the bearing area required is $\frac{24.6}{8} = 3.075$ square feet. The length of the wall is 6 feet and the actual width of bearing required in consequence is $\frac{3.075}{6} = .512$ inch, so that the wall has several times the required bearing on the foundation footing. As the lower wall is 17 inches in thickness and half the thickness is 8.5 inches, and since the distance 8.23 inches from the outside of the wall to the line of action of the pressure so nearly approaches the center line, the wall may be considered as concentrically loaded with respect to the center of the lower wall.

34. The footing area bearing on the soil is next to be proportioned. Provided that the soil will safely sustain $3\frac{1}{2}$ tons per square foot, the bearing area required is $\frac{24.6}{3.5} = 7.029$, practically 7 square feet. From this figure, it is evident that if the footing were only made the same width as the wall, it would have sufficient bearing on the soil for the length of wall under consideration. The length being 6 feet, and the width 17 inches, or 1.4166 feet, this area will amount to $1.4166 \times 6 = 8.499$ square feet, which is excessive.

Though the bearing of the foundation footing on the soil is ample, owing to the fact that the center of pressure is somewhat without the center of the wall, it would be advisable to proportion the footing for an excess of projection on the outside as compared with the projection on the inside. A slight projection for the footing is desired in all instances, from the fact that the stability of the wall is thus increased.

If for any reason it is anticipated that, owing to the center

of pressure being outside of the center of the bearing, the maximum unit pressure on the soil will be greater than the allowable, the maximum unit pressure should be ascertained by formula **1**, *Statics of Masonry*, Part 1. If, after the application of this formula, the maximum unit pressure is found to be excessive, then the footing must be so proportioned that the maximum pressure is reduced.

35. All problems like the above, and in general all calculations for the design of masonry construction, are theoretical, and the actual conditions that exist are only approached. A knowledge of the principles involved in this class of work greatly assists in the design and furnishes landmarks for guidance and checks on the work, which further insure stability and security. The loads on the wall and the resistance of the materials of which the wall is composed will vary so that the results of the preceding solution simply show that the wall is safe, and that the footings are not unreasonably heavy and in consequence uneconomically designed.

Some authorities recommend that the center of resistance be placed slightly outside the center of pressure, so that the wall in settling will be thrown inwards against the floors rather than outwards. If, in the problem explained in connection with Fig. 30, the eccentricity is made 2 inches, the distance from the outside face of the wall to the assumed center of resistance will be 8.23 – 2 = 6.23 inches. When 1 inch projection of footing is allowed on the inside of the wall, one-half of the width of the footing is (17 – 6.23) + 1 = 11.77 inches. The total width of the footing is twice this dimension, or 23.54, and the projection on the outside of the wall is then 5.54 inches. The figure shows clearly these calculated dimensions. These should be adjusted to the dimensions of practice.

36. Where a supporting wall contains many openings of considerable size, much of the weight is carried on piers that concentrate great loads on the foundation. The following problem explains the principles involved in the design of such a wall:

Fig. 32 shows one-half of a 40-foot front, city factory building. There is a row of columns down the middle, supporting a line of girders and the floor is carried by girders and party walls; thus, no load comes on the front from the floor except that due to the reaction from

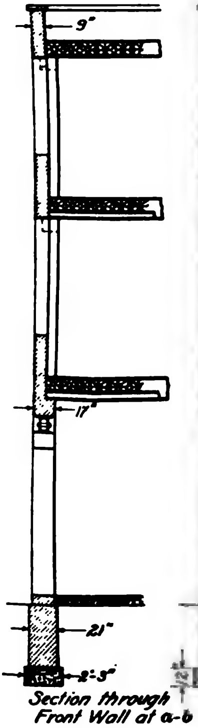


Fig. 31

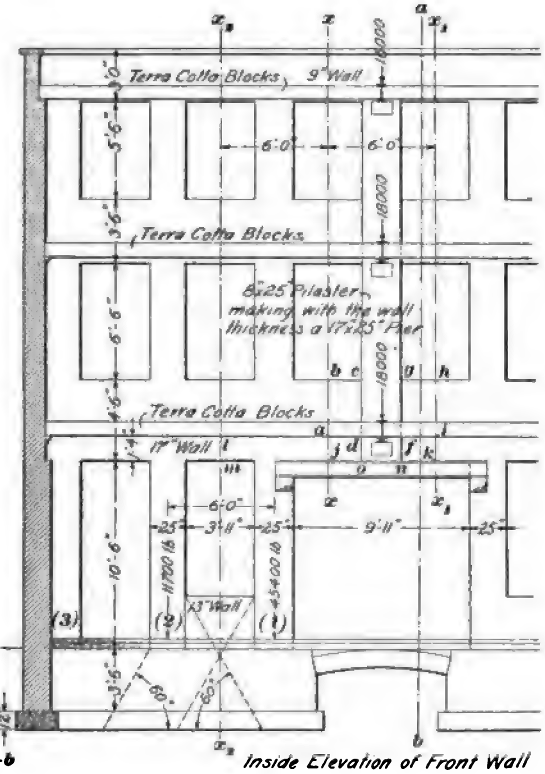


Fig. 32

the end girder in the bay adjacent to the wall under consideration. This load is equal to about 52,000 pounds and is apportioned as follows:

Load from roof	16,000 pounds
Load from third floor	18,000 pounds
Load from second floor	18,000 pounds

By inspection, it is seen that the wall is practically reduced to a series of piers connected at the top and bottom, at each story, thus forming the necessary openings; a cross-section through the wall, showing its thickness at the several stories, is shown in Fig. 31.

The first point to ascertain is the location of the weakest place in the wall, which, it is apparent, is in the piers at the side of the large opening. In commencing the solution of the problem, it is necessary to determine the weight of the brickwork from the top downwards. Therefore, considering the portion of the wall between the lines $x\,x$ and $x_1\,x_1$, Fig. 32, and referring to the section of the wall in Fig 31, the superimposed weight on the third-story pier, which is supported on the steel-beam girder over the opening, may be calculated as follows:

Parapet wall, 3 × 6 × .75 @ 112 .	1512 pounds
Reaction from end of roof girder .	16000 pounds
Total load on third-story pier .	17512 pounds

If the brickwork is laid up in lime mortar and its safe bearing resistance is assumed to be 100 pounds per square inch, the total bearing area required of the pier under consideration is 17,512 ÷ 100 = 175 square inches. The pier, consequently, is sufficiently strong, since it is 25 inches wide, if made 12 inches thick; but a 12″ × 14″ × 8″ bearing plate, or templet, is required under the end of the roof girder; therefore, in order that this may not show on the face of the wall, the pier is made 17 inches thick.

The second-story pier directly beneath the one just analyzed may now be discussed. The weight on it, in pounds, is as follows:

Load from roof girder and parapet wall .	17512
Third-story wall, 3.5 × 3.916 × .75 @ 112 .	1152
Third-story pier, 2.083 × 1.416 × 9 @ 112 .	2973
Load from third-story girder	18000
Total load over second-story pier . . .	39637

As the wall is assumed to be laid in lime mortar, the brickwork has an allowable unit bearing resistance of 100 pounds.

The required area of the pier is 396 square inches. As the area of the pier is 17 by 25 inches, its actual area is 425 square inches, so that it is amply strong for the superimposed load.

The load on the steel-beam girder over the large opening included between the lines $x\,x$ and $x_1\,x_1$ is equal to the weight just calculated supplemented by the weight of the second-story pier, extending from the soffit of the second-story window to the top of the steel-beam girder and the portions of the wall marked $a\,b\,c\,d$, $f\,g\,h\,i$, $j\,a\,d\,o$, and $n\,f\,i\,k$, together with the second-story floor load. These loads, in pounds, are as follows:

Load on second-story pier	39637
Weight of second-story pier, 11 × 2.083 × 1.416 @ 112	3634
Load from end of second-story girder . .	18000
Weight of portions $f\,g\,h\,i$ and $a\,b\,c\,d$, 2 × 3.166 × 1.958 × .75 @ 112	1041
Weight of portions $j\,a\,d\,o$ and $n\,f\,i\,k$, 2 × 1.333 × 1.958 × 1.416 @ 112	828
Approximate weight of steel-beam girder	2280
Total load on steel-beam girder	65420

Since the load on the steel-beam girder between the lines $x\,x$ and $x_1\,x_1$ amounts to practically 66,000 pounds, the weight from the end of the girder coming on pier (*1*) is equal to 33,000 pounds, to which may be added the weight of the 9-inch wall between the lines $x_2\,x_2$ and $x\,x$ and the portion of 17-inch wall designated by $m\,l\,a\,j$. The superficial area of the 9-inch wall between these lines is 6 × 21.66 = 129.996 square feet, and the deduction to be made for window openings from this area is 6.5 × 3.916 + 5.5 × 3.916 = 46,992 square feet. Hence, the total net superficial area of 9-inch brickwork directly supported by pier (*1*) is 129.996 − 46.992 = 83.004 square feet. This area of wall has a thickness of 9 inches, so that its weight is 83.004 × .75 @ 112 = 6,972 lb. The weight of the portion of the 17-inch wall $m\,l\,a\,j$ is 1.333 × 6 × 1.416 @ 112 = 1,268 pounds. The

total weight sustained by pier (*1*) will then be 33,000 + 6,972 + 1,268 = 41,240 pounds.

Because the pier in the first story is subjected to considerable weight and also from the fact that the wall in this story is greatly weakened with window and door openings, the brickwork in this story should be laid in cement mortar. Considering that good brickwork laid in Portland cement mortar will safely sustain 200 pounds per square inch of area, if the pier is made 25 by 17 inches, its safe resistance to crushing will be 85,000 pounds. This demonstrates conclusively that the pier is amply strong, particularly as the pier has not a greater height than six times its least dimension. For convenience in calculating, the total weight on pier (*1*), just obtained, will be considered in round numbers as 42,000 pounds. However, before considering the footings of the pier, the weight down to the top of the foundation must be calculated, though the piece of 13-inch wall between piers (*1*) and (*2*) may be neglected for the present. The approximate weight in pounds at the base of pier (*1*) will then equal

Weight at top of pier	42000
Weight of pier (*1*)	3400
Total weight at top of foundation . . .	45400

By making a detailed estimate, the load on the foundation beneath pier (*2*) is found to be 11,700 pounds. As pier (*3*) is built in with the party wall and sustains little weight besides its own, there is no practical necessity for figuring the weight on the footings under this portion of the wall, especially as footings that will be sufficient for pier (*2*) are adequate for pier (*3*). The center of pressure under pier (*3*) will not be concurrent with the center of resistance but the load is so light, in proportion to the supporting area, that there is little likelihood of unequal settlement and the resulting damage.

37. The dotted lines in the elevation, Fig. 32, show the theoretical distribution of the pressure through the foundations to the soil on which the footings rest. The foundation should not be extended under the large central opening, but

should have an opening and the footings discontinued, the sill of the door resting on an arch. If the foundation is extended beneath the opening, there being no weight above it, this portion of the foundation will not settle to the same extent as the foundation beneath the piers; thus there will be a tendency to throw the piers outwards at the top, and the ground or soil will tend to assume a convex form, which is to be condemned.

38. In deciding on the theoretical lines along which the pressure is transmitted to the footings, the character of the masonry is to be considered; with well-built rubble masonry and brickwork, an angle of 1 to 2, or about 60°, is generally adopted; for cut-stone work a smaller angle will do.

To bring the center of resistance as nearly as possible under the center of pressure from pier (*1*), the foundation wall is extended somewhat at the opening side of the pier. Since the dotted lines in Fig. 32 show the probable distribution of pressure through the foundation to the soil, it is observed that the actions of the two piers are blended and, in consequence, the pressure on the soil becomes more intense at the intersection of these lines.

In order to intelligently design the footings for the wall shown in Fig. 32, it is necessary to employ a graphical method, which will be described in connection with Fig. 33. In view (*a*) is shown the diagrammatical elevation of the base of piers (*1*) and (*2*), and the supporting foundation. The loads at the centers of these piers are equal to 45,400 and 11,700 pounds, respectively. Their location from the center line x_1x_1 is in each case 3 feet, as designated on the drawing. The distance of the force 45,400 pounds from the edge *a* at the right is 2 feet 9 inches, while the force of 11,700 pounds is located at a distance from *b*, or the left end of the foundation bearing area, of 3 feet 11 inches. By applying formula **1**, *Statics of Masonry*, Part 1, the value of k_1, or the pressure per lineal inch at the edge nearest to the center of pressure, for the greater load of 45,400 pounds equals 917 pounds, and k_2 equals 9 pounds, while for the foundation

beneath pier (2), through which is exerted the lesser load of 11,700 pounds, the value of k_1 is equal to 154 pounds and k_2 to 54 pounds.

Before proceeding with the force polygon, the weight of the portion of the 13-inch wall included in the rectangle $o\,p\,q\,r$ and the weight of the 21-inch brick foundation included in the

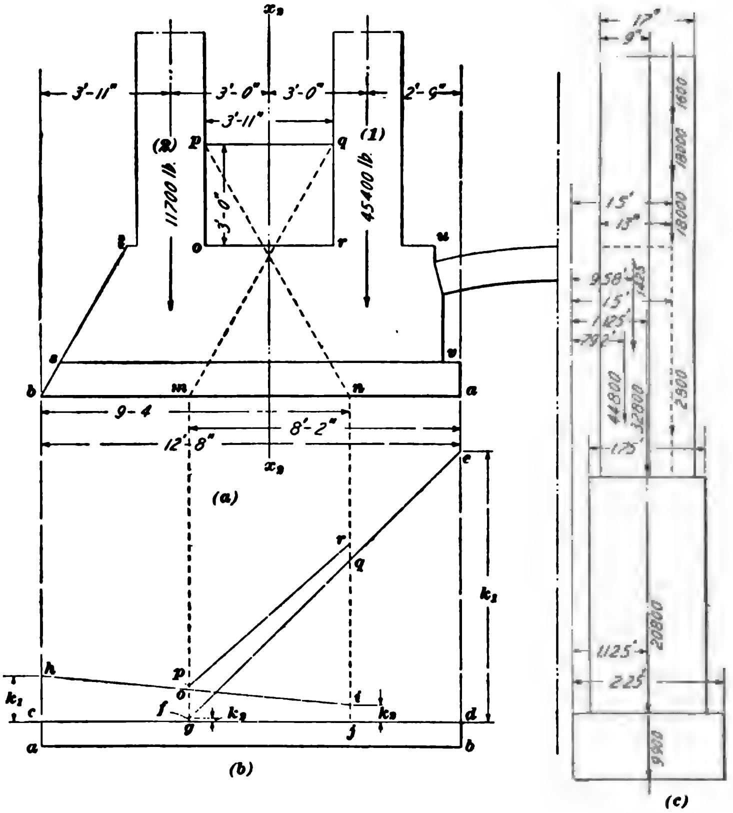


FIG. 33

polygon $s\,t\,u\,v$ and its concrete footing, Fig. 33 (*a*), should be calculated, the vertical reaction of the arch being neglected, from the fact that its action on the foundation soil at *a* is almost annulled by the action of the horizontal thrust of the arch at the abutment. The weights of these two portions,

figured on a basis of 112 pounds per cubic foot for brick and 150 pounds for concrete, are approximately equal to 1,425 and 11,383 pounds, respectively. The sum of these two loads is equal to 12,808 pounds. This load may be considered as uniformly distributed over the footings, so that, as the length of the footing under consideration is equal to 12 feet 8 inches, the pressure per lineal foot on the soil from this weight will be 12,808 ÷ 12.66 = 1,011 pounds, which is 84.2 pounds per lineal inch.

To proceed with the force polygon, lay off the base line *a b* as shown in Fig. 33 (*b*). At either end of it, draw perpendicular lines and lay off *a c* and *b d* to any scale equal to the amount of the unit pressure per lineal inch on the soil from the weight of the foundation and the small portion of brickwork above it, as described in connection with view (*a*). In the case under consideration, the amount of this uniform pressure is 84.2 pounds, so that the rectangle *c d a b* diagrammatically represents a uniform pressure on the soil of 84.2 pounds per lineal inch. From *c d* as a base line, *d e* and *g f* may be laid off to scale equal to the values of k_1 and k_2 for the load on pier (*1*), *d e* being made equal to 917 pounds and *g f* to 9 pounds. Similarly, the polygon *c h i j* may be laid off by taking the values k_1 and k_2 obtained for the load on the foundation from pier (2). In this instance, *c h* is made equal to 154 pounds, while *i j*, to scale, equals 54 pounds.

To obtain, in the force polygon, a representation of the pressure on that portion of the foundation included between *m n* in view (*a*), it is necessary to combine the two polygons just described. Therefore, lay off the distance *f g* from the point *o* and in this manner obtain the point *p*; then lay off the distance *i j* from the point *q*, and obtain the point *r*. By connecting the points *p* and *r*, the polygon described will represent the intensity of the pressure on *m n*, Fig. 33 (*a*). That the pressure is more intense at this point than at any other is due to the blending of the footings for the two piers. The diagram, which now represents the variation of pressure on the footings beneath piers (*1*) and (2), extends from *a* to *h*, *h* to *o*, *o* to *p*, *p* to *r*, *r* to *q*, *q* to *e*, and *e* to *b*. By measuring

any of the ordinates, using $a\,b$ as a base line, the pressure per inch of length on the soil may be determined. It is evident that the maximum pressure on the soil is $917 + 84.2 = 1{,}001$ pounds per lineal inch, and if the footing is assumed

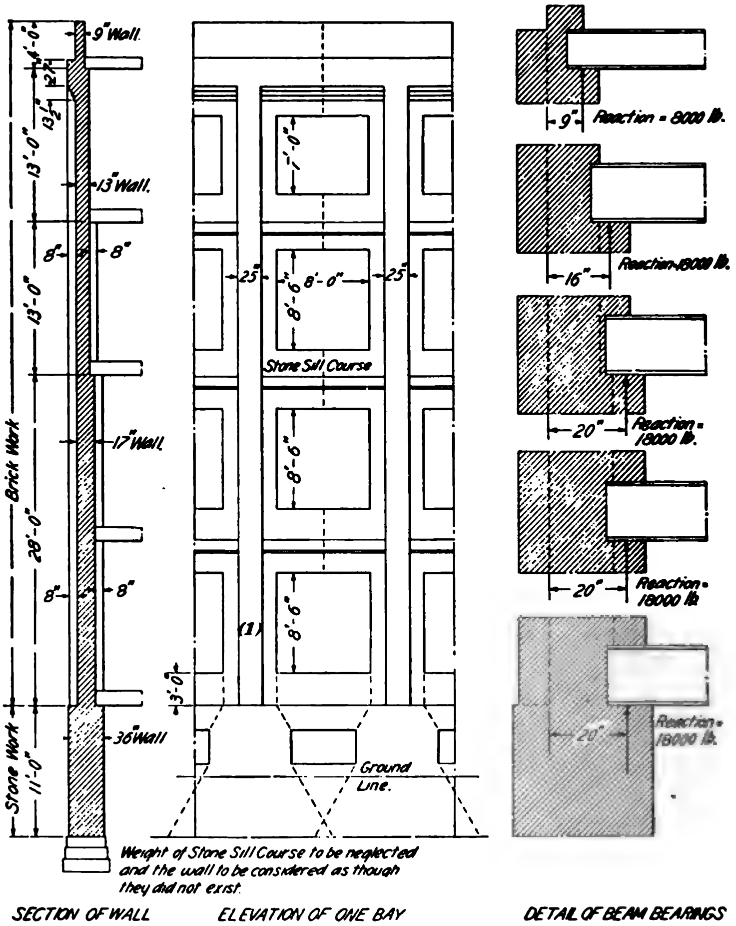


Fig. 34

to have a uniform width of 27 inches, the pressure on the soil is $1{,}001 \div 27 = 37$ pounds per square inch, or 5,328 pounds per square foot. Even a poor foundation soil will

sustain this unit pressure, so that the footing is designed as shown in Fig. 33 (*c*).

The center of this footing area should, if possible, coincide with the center of gravity of all the loads. The calculation for determining, in foot-pounds, this center of gravity, made with reference to Figs. 31, 32, and 33 (*c*), is as follows:

Moment of 9-inch wall	44,800 × .792 =	35482
Moment of 17-inch wall	32,800 × 1.125 =	36900
Moment of 13-inch wall	1,425 × .958 =	1365
Moment of foundation wall . . .	20,800 × 1.125 =	23400
Moment of foundation footing . .	9,900 × 1.125 =	11138
Moment of pilaster over lintel . .	2,800 × 1.5 =	4200
Moment of roof-load reaction . .	16,000 × 1.5 =	24000
Moment of 3d-story load reaction	18,000 × 1.5 =	27000
Moment of 2d-story load reaction	18,000 × 1.5 =	27000
Moment of entire front wall and supported loads	=	190485

Now, by dividing the total moment of 190,485 foot-pounds by the total load of 164,525 pounds, the distance from the outside line of the foundation footing to the center of pressure is found to be 1.157 feet. This dimension is so nearly equal to one-half the width of the footing that it will be sufficient for all practical purposes to make the footings symmetrical about the center line of the wall.

EXAMPLES FOR PRACTICE

1. What will be the maximum width of footing required under the warehouse wall shown in Fig. 34, provided that the soil is assumed to safely sustain a load of 4,000 pounds per square foot? In making the necessary calculation, the brickwork is to be considered as weighing 120 pounds per cubic foot and the stonework 150 pounds per cubic foot, the weight of the footings being neglected. The width of a bay is 13 feet. Ans. 4 ft. 6 in.

2. Determine the position of the center of pressure from the outside face of the foundation wall shown in Fig. 34. Ans. About 1 ft. $8\frac{1}{2}$ in.

3. What will be the unit pressure at the base of pier (*l*), Fig. 34, provided that the brickwork is considered as weighing 120 pounds per cubic foot? Ans. 205 lb.

SUPPORTING WALLS UNDER LATERAL PRESSURE FROM THRUST OF RAFTER MEMBERS

39. Cases frequently occur in building construction in which it is desirable to support the foot of rafter members directly on the wall, a bearing plate being used to distribute the resulting reactions. It is usual to tie the lower ends of the rafters together and in this way resist the outward thrust. In instances, however, where the building is comparatively small, and it is necessary to obtain headroom, the construction shown in Fig. 35 is sometimes employed. This construction, while not to be particularly recommended, is given as an example to illustrate the principles involved.

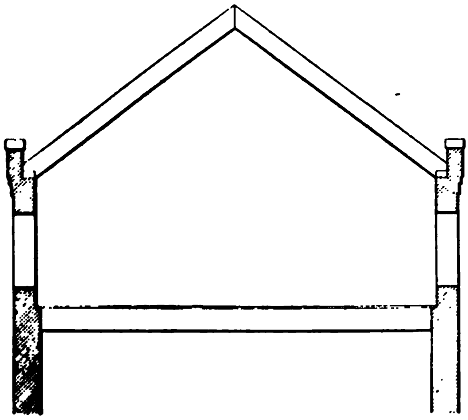


FIG. 35

In considering supporting walls subjected to lateral thrust, the conditions prevailing must be carefully investigated. If

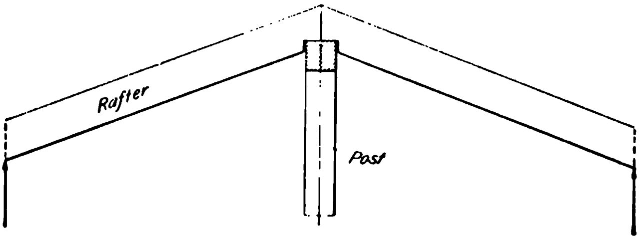


FIG. 36

the rafter members are supported by posts, as shown in Fig. 36, no thrust will be produced, but if the posts support the ridge pole and the rafters are cut against this member,

as shown in Fig. 37, a thrust will be created, provided that the holding power of the nails does not offer a resistance equal to the reaction at this support.

40. In order to make clear the discussion of the analysis of a supporting wall subjected to a lateral thrust, the following example is introduced:

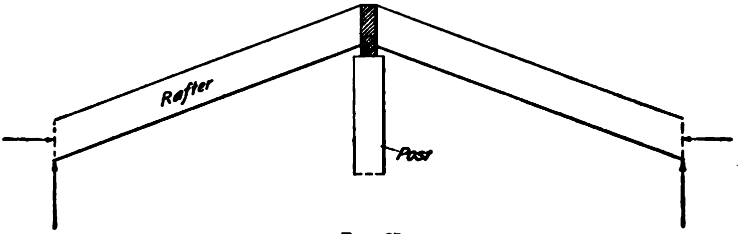


Fig. 37

In Fig. 38 is shown the sectional elevation of a two-story building, the roof of which is supported on two rafter members not in any way tied together at the foot. These rafters are spaced 2 feet from center to center and their foot rests on a wall plate that distributes their thrust uniformly along the length of the wall. The rafter members of the roof sustain a vertical load due to the weight of the roof covering, as well as oblique loads normal to the slope of the roof that are created by the wind pressure. Referring to Fig. 39 (*a*), which may be called the frame diagram of the problem under discussion, it will be observed that the vertical loads *W*, *W* amount to 264 pounds each, while the oblique load *P* due to the wind pressure on the roof is 648 pounds. The vertical reactions are equal to the sum of the vertical loads, or 264 pounds each, which are designated in the diagram by R_1 and R_2. It will be observed that the oblique force, 648 pounds, is located considerably nearer the apex of the

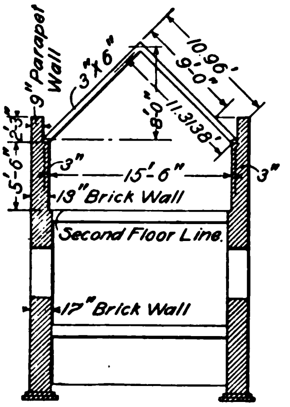


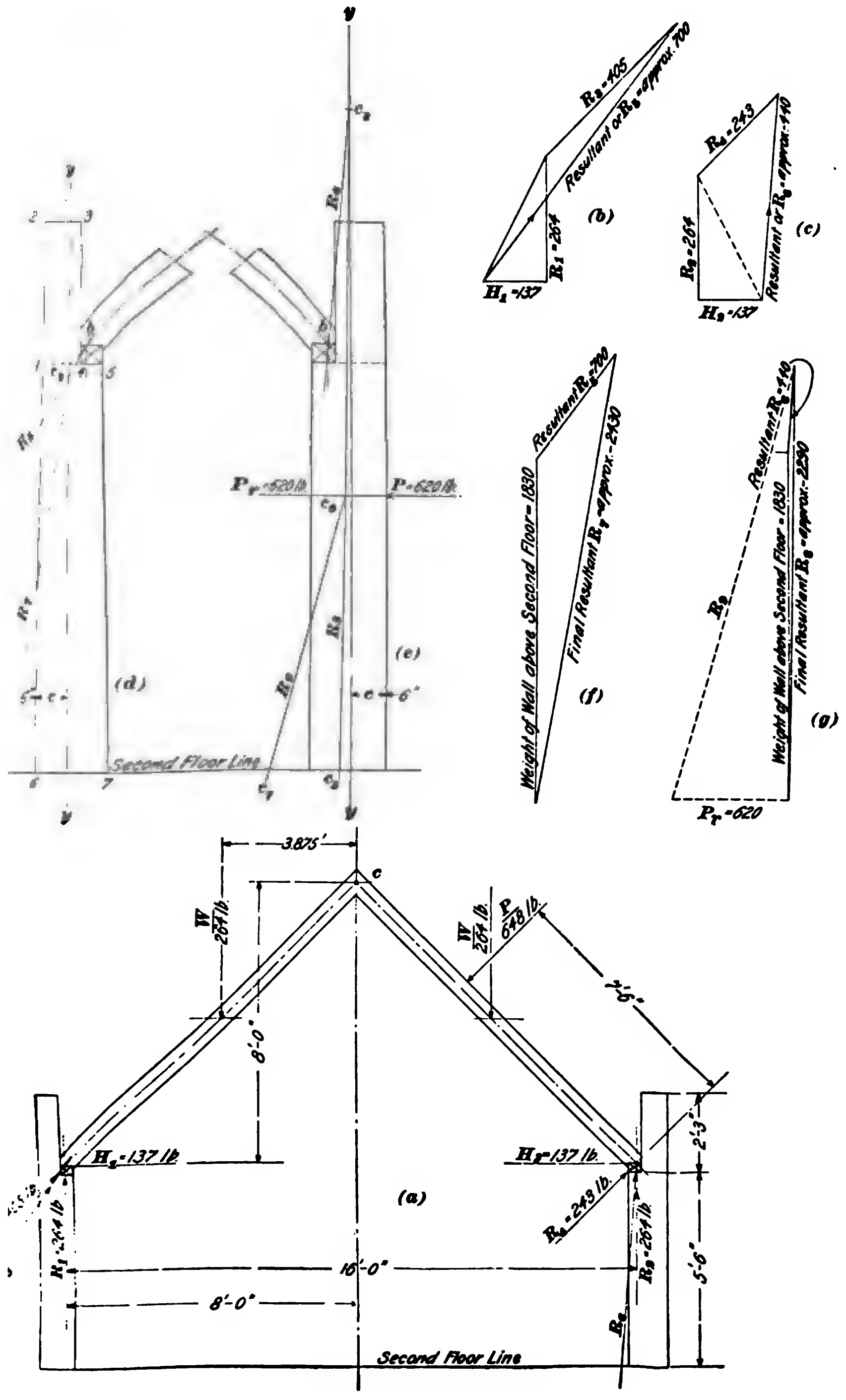
Fig. 38

truss than the foot, on account of the height of the parapet wall that protects the lower portion of the roof from any horizontal wind pressure. The force P produces oblique reactions at the left-hand and right-hand ends of the rafter members, which may be calculated by the principle of moments, and are found to equal 405 and 243 pounds, these values being represented by R_3 and R_4, respectively. The oblique forces instituted by the wind pressure on the truss are in equilibrium of rotation about any point. This is not true, however, of the vertical loads and their corresponding reactions, for they exert horizontal thrusts H_1 and H_2, which are equal at the foot of both the rafter members and are opposed to each other. The amount of the thrust is readily determined by dividing the rise into the algebraic sum of the moments of the vertical forces acting about c as the center of moments. The rise, which is 8 feet, is the lever arm with which H_1 and H_2 act about the center of moments c. The calculation for the amount, in foot-pounds, of either the thrust H_1 or H_2 will then be as follows:

Positive moment . . . . . . $264 \times 3.875 = 1{,}023$
Negative moment . . . . . . $264 \times 8 = 2{,}112$
Algebraic sum of moments . . . . $= 1{,}089$

so that, $H_1 = 1{,}089 \div 8 = 137$ pounds

The three reactions at the foot of each rafter member have now been determined. For the left-hand foot they are R_1, R_3, and H_1, and for the right-hand R_2, R_4, and H_2. A single force may be obtained at each support that will equal, in effect, these reactions, and this single force will be their resultant. To obtain the resultant reactions, or R_5 and R_6, draw the polygons of forces shown in Fig. 39 (*b*) and (*c*). In each of these polygons, the resultant is obtained by laying off, to any scale of unit measurement equal to a number of pounds, the three reactions extending in their respective directions; the resultant is the line that connects the extremes of the first and last forces. Further observation of these force polygons, Fig. 39 (*b*) and (*c*), demonstrates that in each instance an oblique force may have been substituted



(a)
(b)
(c)
(d)
(e)
(f)
(g)
Second Floor Line
P_T = 620 lb.
P = 620 lb.
R_2 = 405
Resultant or R_6 = approx. 700
R_1 = 264
H_1 = 137
R_6 = 243
R_3 = 264
H_2 = 137
Resultant or R_6 = approx. 440
Resultant R_6 = 700
Weight of Wall above Second Floor = 1830
Final Resultant R_7 = approx. 2430
Resultant R_6 = 440
Weight of Wall above Second Floor = 1830
Final Resultant R_8 = approx. 2290
P_T = 620
3.875'
W 264 lb.
W 264 lb.
P 648 lb.
8'-0"
2'-6"
2'-3"
5'-6"
16'-0"
8'-0"
H_1 = 137 lb.
H_2 = 137 lb.
R_1 = 264 lb.
R_6 = 243 lb.
R_3 = 264 lb.
Second Floor Line

Fig. 39

for the vertical and horizontal forces, when the resultant reactions R_5 and R_6 would have been the same. Therefore, in laying off the polygon of forces, it is not necessary to lay off the several reactions in any particular sequence or order.

41. Now, that a single reaction at the foot of the rafter members has been determined for each abutment, the next step is to find what effect they will have on the walls of the building above the second-floor line, for at this level, the walls may be considered as securely tied together by the floorbeams and have sufficient lateral resistance to prevent the overturning of the wall at any point below this level The pressure at the second-floor level that tends to overturn the walls consists of the resultant of the actions R_5 and R_6 just found in direction and amount and the weight of the portion of the wall above, acting along a vertical line passing through the center of gravity of the wall.

The direction and amount of this resultant that acts on the wall may be obtained by drawing two polygons, using the weight of the wall for one force and the final reaction at the abutments determined by the polygons in Fig. 39 (*b*) and (*c*) for the other. The force polygon at the left abutment is shown in Fig. 39 (*f*); a similar polygon for the right support is shown in view (*g*). In each of these polygons, the vertical force is laid off to equal the weight of that portion of the wall above the second-floor line included between the centers of two rafter members. The resultant reactions R_5 and R_6 are drawn parallel with these forces determined by the force polygons in Fig. 39 (*b*) and (*c*), and are made equal in direction and amount to these forces. It will be noticed, however, that in the figure it was necessary, on account of limited space, to draw the force polygons, Fig. 39 (*f*) and (*g*), to a different scale from that used in constructing the force polygons in views (*b*) and (*c*). When, in each of these diagrams (*f*) and (*g*), the vertical force equal to the weight of the wall and the resultant reactions R_5 and R_6 have been drawn, the final resultants

R_7 and R_8 may be described. These lines give the amount, by scale, and the direction of the forces that tend to overturn the walls. Before proceeding further with the solution of the problem, it is necessary to draw sections of the walls to a somewhat larger scale than is shown in Fig. 39 (*a*), so that views (*d*) and (*e*) are made to a scale twice as large.

42. The next step is to determine the position of the vertical line $y\,y$ that passes through the center of gravity of the wall section. To determine its distance from the outside face of the wall, assume that this face of the wall is the origin of moments and calculate the distance c, as described in *Properties of Sections*. The calculations for the distance c are as given in the following tabulation:

Portion in Feet	Thickness in Feet		Height in Feet		Distance Center of Gravity in Feet	Moment of Portion in Foot-Pounds
1-2-3-4,	.75	×	2.25	×	.375 . . .	.632
1-5-6-7,	1.083	×	5.5	×	.5417 . .	3.226
Total moment of section equals					. . . .	3.858

The total area of the section is equal to 7.64, so that the distance c is $3.858 \div 7.643 = .504$ feet. Since this result is not $\frac{1}{16}$ inch in excess of 6 inches, the distance c will be considered as 6 inches, so that the line $y\,y$ in both (*d*) and (*e*) may be drawn 6 inches from the outside face of the wall. To determine the points from which R_7 and R_8 should be drawn, it is necessary to draw through the section from the point of bearing at the abutment, the reactions R_8 and R_9 until they intersect the line $y\,y$ in both (*d*) and (*e*). By drawing these reactions in this manner from the points b, the points c_1 and c_2 are found. From these points draw the indefinite lines R_7 and R_8 parallel with the final resultants obtained in the diagrams (*f*) and (*g*); the line R_7 corresponds with the resultant R_7 in Fig. 39 (*f*), while the line R_8 is parallel with the line thus marked in Fig. 39 (*g*). These lines, extended, intersect the second-floor line at the points c_3 and c_4.

Since the resultant R_7 falls far beyond the outside edge of the wall, the left-hand wall is undoubtedly unsafe when

the adhesion of the mortar is not considered, but the right-hand wall is entirely stable from the fact that the point c_s falls well within the middle third of the thickness of the wall; but in considering the right-hand wall, which is the windward wall of the building, a wind pressure against its surface similar to the wind pressure on the roof was not considered, and if the building were constructed in the open, that is, not sheltered by adjacent buildings, a wind pressure against the vertical face of the wall should enter into the discussion. This wind pressure, which, referring to Fig. 39 (*e*), will be designated as *P*, acts at the middle point of the wall between the second-floor line and the top of the parapet, and figuring in this case on a pressure of 40 pounds per square foot, amounts to 620 pounds. The reaction opposed to this wind pressure could be represented by P_r of equal amount. By the introduction of this wind pressure, there is obtained on the right-hand wall a new resultant, whose direction and amount may be found from the force polygon in Fig. 39 (*g*). To determine this resultant, lay out, by scale, the reaction P_r in the diagram, when, by connecting the extremes, the line R_s is found. R_s drawn from c_s in view (*e*), intersects the second-floor line at c_7, which is a considerable distance without the base of the wall.

43. In summing up the discussion of the construction shown in Fig. 39 (*a*), it is apparent that under that condition of loading and wind pressure the building is unsafe. There might be circumstances, however, that would prevent the destruction of the walls by the thrust of the rafters; for instance, the walls might have been constructed of first-class materials, with the best workmanship when a high adhesive strength of the cement or mortar would have been obtained, and, in consequence, the wall would be able to resist a considerable amount of tension on the leeward side. Again, the wall might not be of any considerable length and the building might be provided with masonry end walls or heavy partition walls. By these means, the side walls would be greatly strengthened, for portions of the partition and end

walls would be so incorporated with the side walls as to form a part of them and thus tend to move the line yy passing through the center of gravity of the wall toward the interior of the building. At least such would be the case with the section of the wall shown in view (d), while in the section of the wall shown in view (e) the end walls and

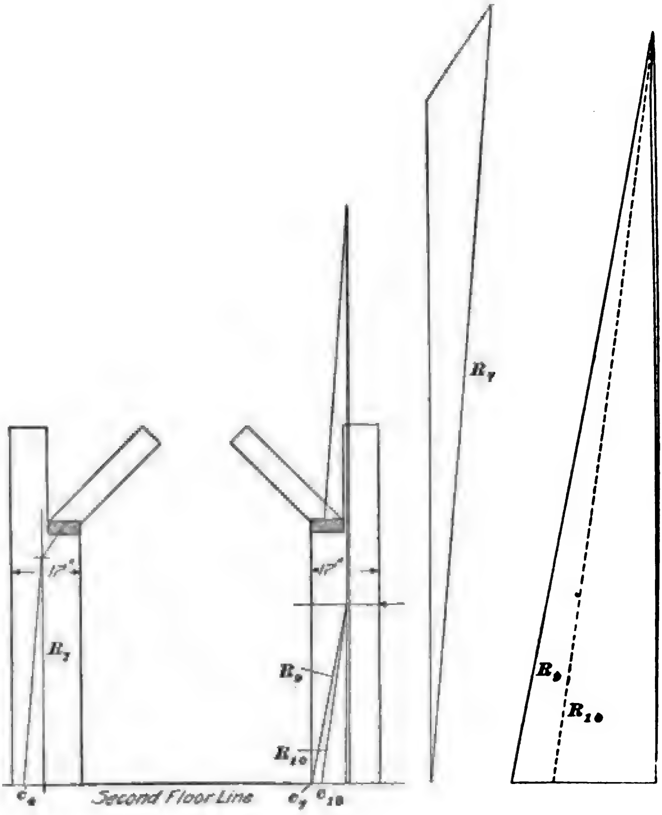
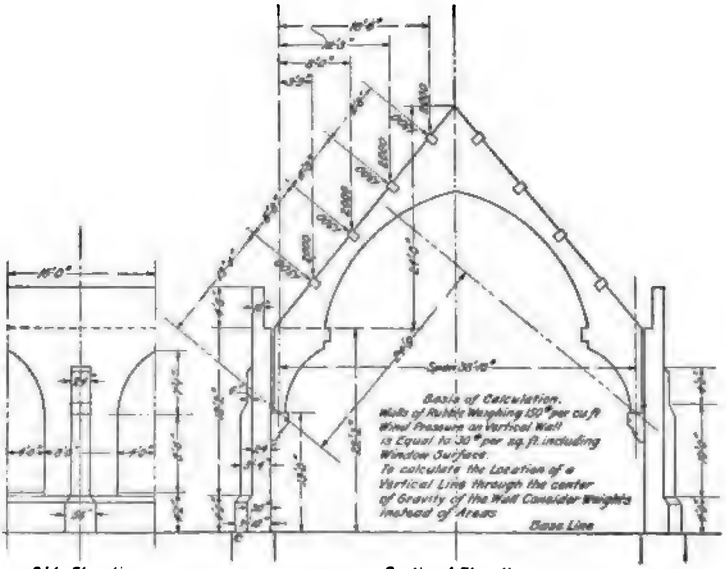


FIG. 40

partition walls would act as buttresses and produce counter-thrust to the pressure P, in this way changing the obliquity of R_8 and causing it to assume more nearly the vertical.

Since the construction shown in Fig. 39 may be regarded as insecure, if not positively unsafe, when the vertical loads

on the rafter members are augmented by a heavy wind pressure, the question arises as to how the stability of the building may be secured. It is assumed that the headroom to the peak of the roof above the second-floor line is absolutely required so that the interposition of a collar beam or tie to bind the two rafters together is precluded. The walls must therefore be increased in thickness, for by changing the construction in this manner the walls are given a broader base and their weight is greatly increased. Also,



Side Elevation Sectional Elevation

FIG. 41

since brickwork does not weigh as much as stonework, the material may be changed to rubble, particularly as the wall has been thickened.

The walls as remodeled are shown in Fig. 40. Here they are made 17 inches in thickness and have been considered as constructed of rubble masonry, weighing 150 pounds per cubic foot. By constructing the usual diagrams and obtaining R_7 and R_8, it will be noticed that these lines intersect the second-floor line at the points c_4 and c_7 at least within the

edge of the wall, making the left-hand wall appear entirely stable and the right-hand wall likewise safe, provided that it may be depended on to resist a slight amount of tension. It is admissible, also, that the unit wind pressure of 40 pounds, originally adopted, is high and that with reasonable safety it may be reduced to 30 pounds. When the wind pressure is taken at this figure, the direction of the final pressure on the right-hand abutment will be approximately that of R_{10}, Fig. 40, demonstrating that under this assumption the wall, as redesigned, is amply secure.

EXAMPLES FOR PRACTICE

1. Determine the amount and direction of the resultant reaction on the left-hand wall due to the horizontal thrust, the vertical reaction due to the load on the roof, and the oblique reaction due to the wind pressure on the roof of the Gothic construction shown in Fig. 41. It is assumed that the entire horizontal thrust of the truss is resisted by the side walls. The distance from the center of pressure on the bracket to the inside line of the wall is 12 inches.

Ans. { Amount, about 8,500 lb.
Direction, about 15° from vertical

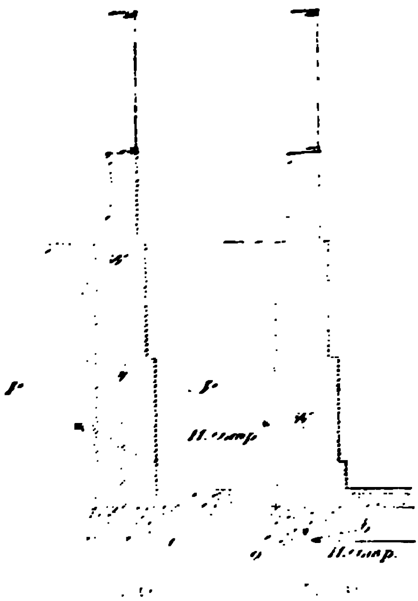
2. Referring to Fig. 41, at what distance from the outside edge c of the buttress will the final resultant of the oblique reaction, the weight of the wall, and the wind pressure on the wall intersect the base line?

SUPPORTING WALLS UNDER LATERAL PRESSURE FROM EARTH

44. Supporting walls subjected to a lateral thrust from earth are usually foundation walls enclosing a cellar or basement. It is not necessary for such walls to be as massive as retaining walls because the superimposed weight, due to the floors and walls, adds to the resistance of the foundation wall. This fact is illustrated in Fig. 42, in which the overturning moment of the pressure P about the center of moments c is resisted by the moment of the superimposed load W acting about the same point.

In the design of such walls, it must always be remembered that the weight of the superimposed wall and floors is not available until the building is practically completed. When,

[illegible]



[illegible] light nature only the [illegible] these walls support [illegible]

15. [illegible] foundation wall subjected to [illegible] the bed of the footing [illegible] the stability against sliding [illegible] the superimposed weight. In the design of the footings [illegible] therefore, the bed of the footing [illegible] sloped upwards from the earth's pressure. [illegible] frequently done in the construction of retaining walls. A footing with a sloping bed is shown in Fig. 43. This type of footing under a foundation wall would be dangerous

unless the horizontal component of the action of the weight W on the inclined plane ab was equalized by the horizontal component of the pressure from the earth filling P. The tendency of a footing constructed in this manner beneath a foundation wall, in which the weight on the wall is great in proportion to the lateral thrust of the earth filling, is to slide outwards and cause the wall to bulge inwards, but this tendency is partially prevented by the girders, beams, and floor construction. Consequently, should the conditions with regard to the character of soil and the proportion of the lateral pressure to the weight superimposed be such as to indicate a marked tendency to slide the footing inwards, and thus cause the wall to bulge outwards—a dangerous possibility—the footing may, with advantage, be sloped as indicated in Fig. 43.

The footings for foundation walls subjected to the lateral pressure of earth should in all cases be built well below the cellar floor, for in this way an additional security against sliding is obtained.

46. The method employed in the design of foundation or supporting walls subjected to lateral pressure is only a **method of verification.** By this it is meant that the dimensions of the wall or structure are assumed and the method is employed to determine its stability. Therefore, in the design of foundations or supporting walls subjected to lateral pressure, the wall must first be proportioned, the loads or weights on it approximately determined, and the location of their centers of effort found. Likewise, the direction and amount of the probable lateral thrust from the earth, and the location of its point of application, must be either assumed or determined by the rules and formulas commonly applied in the practice of this branch of engineering. These rules and formulas are described in *Heavy Foundations* and *Retaining Walls*.

47. In order to explain each step in the determination of the stability of a supporting wall subjected to lateral pressure, the following problem is discussed:

In Fig. 44 is shown a section of the wall of a warehouse having fireproof floors designed to support 300 pounds per square foot of floor surface. The weight of the floor itself is 70 pounds per square foot and it is so constructed as to span the distance between the wall and the central columns *a* without the introduction of beams or girders extending across the building. In this way the floor construction bears directly on the wall adjacent to the inside edge, and the weight transmitted to the wall is uniform for each lineal foot of length. Both the foundation and the wall of the building above finished grade are to be of brick. The roof construction is to be similar to the floors, and weighs about 50 pounds per square foot.

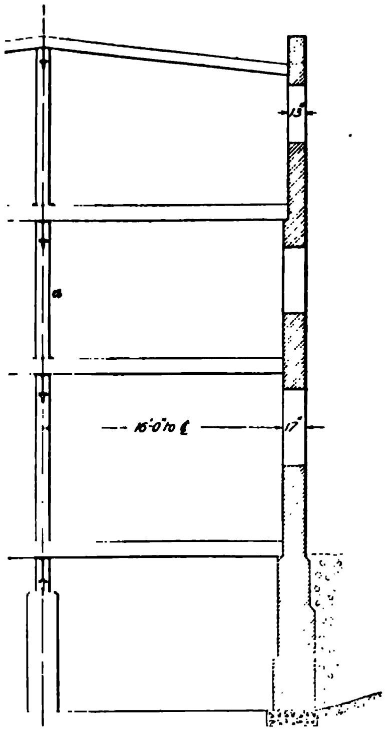


FIG. 44

In the solution of the problem, the live loads should not be considered because of the uncertainty of their resistance, so only the dead loads of the floor are to be included in the weight on the foundation wall that assists in the resistance of the lateral thrust exerted by the earth filling. When the floor and wall loads, in pounds, on the foundation wall have been calculated, they are found to be as follows, the estimate being based on 16 lineal feet of the wall and floors, or one bay:

Load from roof construction . .	107000 pounds
Load from three floors . . .	27200 pounds
Load from 13-inch wall	19800 pounds
Load from 17-inch wall	51000 pounds
Total load	108700 pounds

The position of these several loads may be observed from Fig. 45 (*a*), so that only the location of their resultant, or sum with reference to the face of the wall, is to be determined by calculation. The roof and floor loads are each considered as being applied 3 inches from the inside face of the wall that supports them. Thus, the center of action of the roof load is located 10 inches from the outside face of the wall, while the floor loads may be considered as being applied 14 inches from the outside face of the wall. The centers of action of the superimposed walls lie in a line drawn through their respective centers of gravity. The action of the 13-inch wall, in consequence, is concentrated along a line $6\frac{1}{2}$ inches from the outside face, while the weight of the 17-inch wall is applied $8\frac{1}{2}$ inches from the outside face of the wall.

To determine the position of the resultant of all these loads, or, in other words, the point at which a single reaction equal to their sum would be applied in order that equilibrium of rotation about the base of the wall could be maintained, the sum of the moments of each of these loads about the outside face of the wall must be divided by the sum of the loads. The calculation for the position of R will, therefore, be as follows:

$$
\begin{aligned}
10{,}700 \times 10.0 &= 107000 \text{ inch-pounds} \\
27{,}200 \times 14.0 &= 380800 \text{ inch-pounds} \\
19{,}800 \times 6.5 &= 128700 \text{ inch-pounds} \\
51{,}000 \times 8.5 &= 433500 \text{ inch-pounds} \\
\text{Sum of moments} &= 1050000 \text{ inch-pounds}
\end{aligned}
$$

$$1{,}050{,}000 \div 108{,}700 = 9.65 \text{ inches, the required distance}$$

48. The amount and position of the final resultant of these several loads having been found, the solution of the problem may be completed by laying out the diagrams shown in Fig. 45 (*b*) and (*c*). Before the diagram (*b*) can be drawn, it is necessary to obtain the amount, direction, and position of application of the two forces acting on the foundation wall. These are the total vertical force due to the combined weight of the foundation and the superimposed wall, and the lateral force due to the thrust of the earth's pressure.

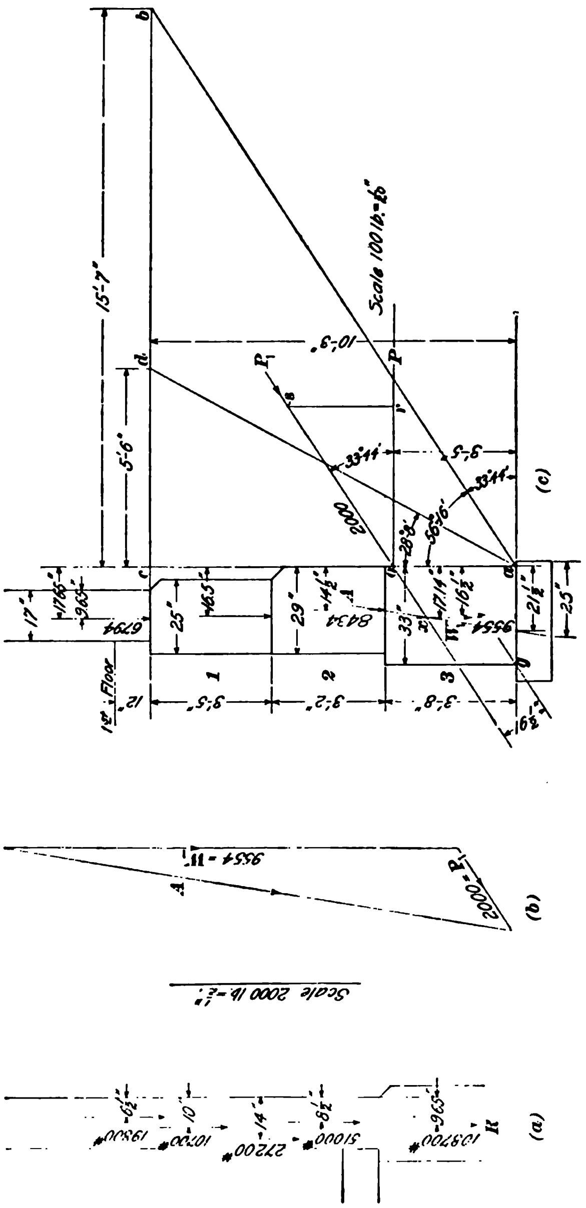


FIG. 45

The weight of the superimposed wall for a length of 16 feet has been found to equal 108,700 pounds. This length was at first considered as a unit from the fact that it is the length of one bay and that by taking such a portion of the building all window openings and pilasters may be considered in the calculation. In the solution of all problems like this, it is usual to analyze a section of the wall 1 foot in length so that in this way all measurements of area on the section will express volume, and consequently the weight may more readily be found. The diagrams shown in Fig. 45, therefore, illustrate a section through the wall and filling 1 foot in length.

The superimposed weight W on the foundation shown in Fig. 45 (*c*) will be $108,700 \div 16 = 6,794$ pounds, approximately. This vertical load, or force, is increased to a considerable extent by the weight of the portions of the foundations marked *1*, *2*, and *3*, which weigh, approximately, 790, 850, and 1,120 pounds per foot of length, respectively; therefore, the total weight W_1 coming on the footing will equal the sum of these weights and the load W, or 9,554 pounds.

It is now necessary to determine the point of application of this pressure W_1, which may be accomplished by the same method as that employed in calculating the position of the reaction R in Fig. 45 (*a*). Taking the moments, in inch-pounds, of the several weights about the outside-face line of the wall, they may be tabulated as follows:

Moment of W	$6,794 \times 17.65 =$	1 1 9 9 1 4
Moment of portion *1* .	$790 \times 16.50 =$	1 3 0 3 5
Moment of portion *2* .	$850 \times 14.50 =$	1 2 3 2 5
Moment of portion *3* .	$1,120 \times 16.50 =$	1 8 4 8 0
Sum of moments	=	1 6 3 7 5 4

Since the total weight is 9,554 pounds, the distance that the total pressure W_1 is located from the outside face of the wall must be $163,754 \div 9,554 = 17.14$ inches.

49. The vertical force, its location and amount having been determined in this manner, it remains to find the amount

of the lateral thrust of the earth filling and then to compare the moment of the former force with that of the latter about the point g, the vertical force being regarded as creating a positive moment about the point while the lateral thrust causes a negative moment about the same point.

The point of application of the earth's pressure and its direction, as well as its amount, may be determined by the formulas, rules, and methods usually employed in the analysis of the stability of retaining walls. In practice, it is customary to consider the pressure P_1, Fig. 45 (c), as applied at one-third the depth of the filling from the base of the wall. As the depth of the earth filling is 10 feet 3 inches, or 123 inches, one-third of this distance is 41 inches above the base line $g\,a$ of the wall.

The direction of the lateral thrust should coincide with a line drawn at the angle of repose of the material with the line P, which is at right angles with the face of the wall. The angle of repose of any material is the angle or slope that the material naturally assumes when loosely piled, and the earth filling in this problem is considered to be dry and to have a slope of repose of 33° 44′ with the horizontal.

The amount of the pressure P_1 may readily be obtained by laying out and measuring the oblique component of which the pressure P is the horizontal component. To find the amount of this component, it is necessary to draw a line representing the angle of repose of the material from the bottom outside edge of the wall, thus determining the distance $c\,b$, and to bisect the angle $c\,a\,b$ by the line $a\,d$, thus determining the position of the point d and, in consequence, the distance $c\,d$. When these several factors have been found, the amount of pressure P may be determined by the formula

$$P = \frac{W\,c\,d}{h} \qquad (1)$$

in which W = weight of triangular prism of earth included in section $a\,c\,d$;

h = height of wall, in feet,

The value of cd in the present example, found from the diagram, is 5 feet 6 inches, so that the weight of the prism of dry earth included in acd and 1 foot in length is

$$\frac{5.5 \times 10.25}{2} \times 110 = 3{,}100$$

provided that the dry earth, well rammed, is considered as weighing 110 pounds per cubic foot.

The horizontal or lateral pressure of the earth P may be found by substituting the several values in the formula, when $P = \frac{3{,}100 \times 5.5}{10.25} = 1{,}663$ pounds. By laying off this amount, 1,663, on the line of action of the pressure P from its point of application on the wall at o, and describing the line from r upwards until the intersection s on the line P_1 is obtained, the amount of P_1 will be known by measuring os with the same scale to which or was laid off. By measuring the line os, the amount of the pressure P_1 is found to be, approximately, 2,000 pounds.

It now remains to determine the direction and amount of the resultant pressure created by the vertical pressure W_1 and the lateral pressure P_1. The intersection of these two forces is at the point x, and the final resultant A is drawn from this point and parallel with the line so marked in the force polygon, Fig. 45 (b), which is drawn by the usual method.

When the final resultant has been drawn as shown in Fig. 45 (c), it is found to intersect the base line of the footing about 25 inches within the outside edge of the footing, and the resultant pressure passes through a point on the upper bed of the footing at least 21½ inches from the outside edge of the wall. Thus, the line of pressure is included in the middle third of the width of both the bed of the wall and the footing, so that there is no possibility of the wall being insecure, provided that the allowable bearing value of the masonry or the soil is not exceeded.

50. The factor of safety against the overturning of the wall about the point g, based on the assumption that the edge

included between the points $a\,c\,d$, and the live load to be added to the weight of the filling is that supported on the area between c and d.

EXAMPLE FOR PRACTICE

In Fig. 46 is shown a section through a foundation wall on which is to be erected a church edifice. Owing to the lack of funds, it is agreed to proceed no further with the structure than the top of the basement band course shown in the view at a, and to provide a temporary roof over the cellar, or basement, so that this portion of the building may be used. The question arises as to whether the foundation wall is of sufficient strength to resist the thrust of the dry earth filling, having a slope of repose of 33° 44′. The superimposed load from the roof is 500 pounds, the weight of the filling is 110 pounds per cubic foot, and the rubble wall, including the stone band course, may be considered as weighing 150 pounds per cubic foot, the projecting portions of the band course being disregarded: (*a*) What distance inside of the edge g of the wall will the line of action of the final pressure intersect the base line of the wall? (*b*) What factor of safety will exist in the wall?

Ans. { (*a*) About 14 in.
(*b*) About 4

BUTTRESSED WALLS

52. In large buildings, such as armories, gymnasiums, auditoriums, and churches, the walls are frequently of great height, practically unsupported by transverse systems of flooring, and commonly supporting roof trusses of long span. It is usual to provide against their overturning from the oblique or horizontal wind pressure on the truss or wall by providing *buttresses* along the exterior face of the wall opposite the support of each truss. These buttresses must be thoroughly bonded and incorporated into the wall masonry, so that they will act in unison with the wall. When properly designed and constructed they reenforce the wall and prevent bulging, besides supplying additional mass to resist the oblique or horizontal forces and change the center of gravity of the wall so that its overturning moment is considerably increased. They likewise influence the location of the center of pressure on the foundation soil.

In the design of **buttressed walls**, it is necessary to first determine the direction and amount of the oblique or

horizontal forces acting on the wall and to find the center of gravity of the combined wall and buttress, so that the action of the weight of the wall and buttress along this line and its influence on the oblique or horizontal actions on the wall may be determined. When the final resultant action on the buttressed wall has been determined in amount and direction, the bearing area at the different sections of the wall and the footings on the soil may be investigated to determine whether the maximum allowable pressure on the masonry or the soil has been exceeded. If the final action from all the forces does not fall within the base of the wall, it is evident that the wall is not in stable equilibrium under the imposed conditions, and either some means must be taken to modify the oblique or horizontal actions due to the wind, or the wall and buttress must be redesigned so that a greater mass of masonry will be provided.

The problem of the design of buttressed walls is an application of the principles explained throughout this Section. The method employed for determining their stability is one merely of verification. It is preferable, after the section of the building has been decided on, to apply a method of verification that is entirely graphical, for the conditions of the problem are best realized and are more immediately brought to the attention of the designer by this method. As the various principles involving the design of a buttressed wall have been explained, and as the problem is best understood by practical illustration, the following example is discussed at length:

53. In Fig. 47 (*a*) is designated the transverse section of a gymnasium that is to be erected for a large university. The clear distance between the walls is 100 feet 6 inches, while the height of the lower chord of the truss from the basement floor level is 47 feet 3 inches. The roof is supported on a Howe truss having a clear span of 104 feet 6 inches, and a rise from the lower chord to the apex of 21 feet. The truss is divided into twelve equal panels, at which the purlins are secured and the roof loads concentrated. Besides the dead

load due to the weight of the roof, the truss is required to support at the second panel point from the ends an additional load due to the suspension rods necessary for the support of the visitors' gallery. The truss is to be constructed of steel and, owing to the length of the span, will have rollers at one end. Therefore, in considering the problem, the truss is assumed as having the right-hand end fixed and the left-hand end on a roller or hinged bearing. The truss, besides being subjected to these dead loads, must sustain the usual oblique wind pressure on either slope of the roof.

It is necessary in the discussion of such a problem to determine the conditions of the truss load, in order to find the reactions at the ends of the truss. These reactions, together with the wind pressure on the vertical surface of the exposed wall, are the forces that tend to overturn the supporting walls. The first step in the problem, therefore, consists in the determination of the actions or reactions at the ends of the truss. The vertical loads on the truss that supply vertical reactions at the ends do not tend to affect the overturning of the wall, but add increased weight to the wall and assist in changing the direction of the oblique action of the wind. Consequently, in finding the direction and amount of the actions at the ends of the truss, both the dead and the wind loads on the frame are considered at once. These actions may be either calculated or determined by graphics; in this discussion, the latter method will be pursued.

The greatest overturning moment is created in the right-hand wall, from the fact that it supports the fixed end of the truss. The actions from the end of the truss sustained by this wall must therefore be found for the wind from the left and the wind from the right. Assuming the wind to be from the left, the load line in the force polygon, Fig. 47 (*b*), may be laid out. In considering the loads on the truss, they may be designated on the load line in any order, though it is convenient to follow some system. In this instance, the dead loads AB'', BC'', CD'', DE'', EF'', FG'', and GG' were first laid off to scale. Then the wind loads shown in the frame diagram, Fig. 47 (*a*), were laid off in the order of $B''B$, $C''C$,

$D'' D$, etc. From the termination of the load line in the force polygon representing the wind loads, the vertical loads on the right-hand portion of the truss were laid off in the order of $G' F'$, $F' E'$, etc. When the load line is drawn in this manner, any pole o is selected and the rays drawn from this point to the divisions on the load line. When the force polygon has been described in this manner, a line drawn from a to a_1 will determine the direction of the reactions of the truss, provided that both ends are fixed. Their amount, however, will not be determined until the equilibrium polygon has been drawn.

54. In this solution, the drawing of the equilibrium polygon is commenced at the left-hand end of the truss. From the heel joint connection c_1 in (*a*), a line is drawn parallel with the ray *1* in the force polygon until it intersects the line of action of the load $A B''$ extended. From this intersection, line *2* in the equilibrium polygon is described parallel with the line so marked in the force polygon until it intersects the line of action of the load $B C''$. In this manner the drawing of the equilibrium polygon is continued to *7*, which intersects the extended line of action of the vertical load supported at the apex of the truss. From the intersection of this line with the line *7*, the line *8* is drawn parallel with the ray *8* in the force polygon. This line *8* in the equilibrium polygon is drawn so that it intersects the line of action, extended, of the first wind load on the left-hand portion of the truss. It is necessary to revert in this manner in drawing the equilibrium polygon, from the fact that the ray *8* in the force polygon is drawn from the intersection of the apex load and the first wind load, for, in drawing the equilibrium polygon from the force polygon, it must constantly be remembered that the lines of the equilibrium polygon must be described in the same order in which the loads occur in the force polygon. When the intersection of the line *8* with the line of action of the first oblique force is determined, the lines *9*, *10*, *11*, *12*, *13*, *14*, etc. may be drawn parallel with their respective rays in the force polygon. From the intersection of the line *13*

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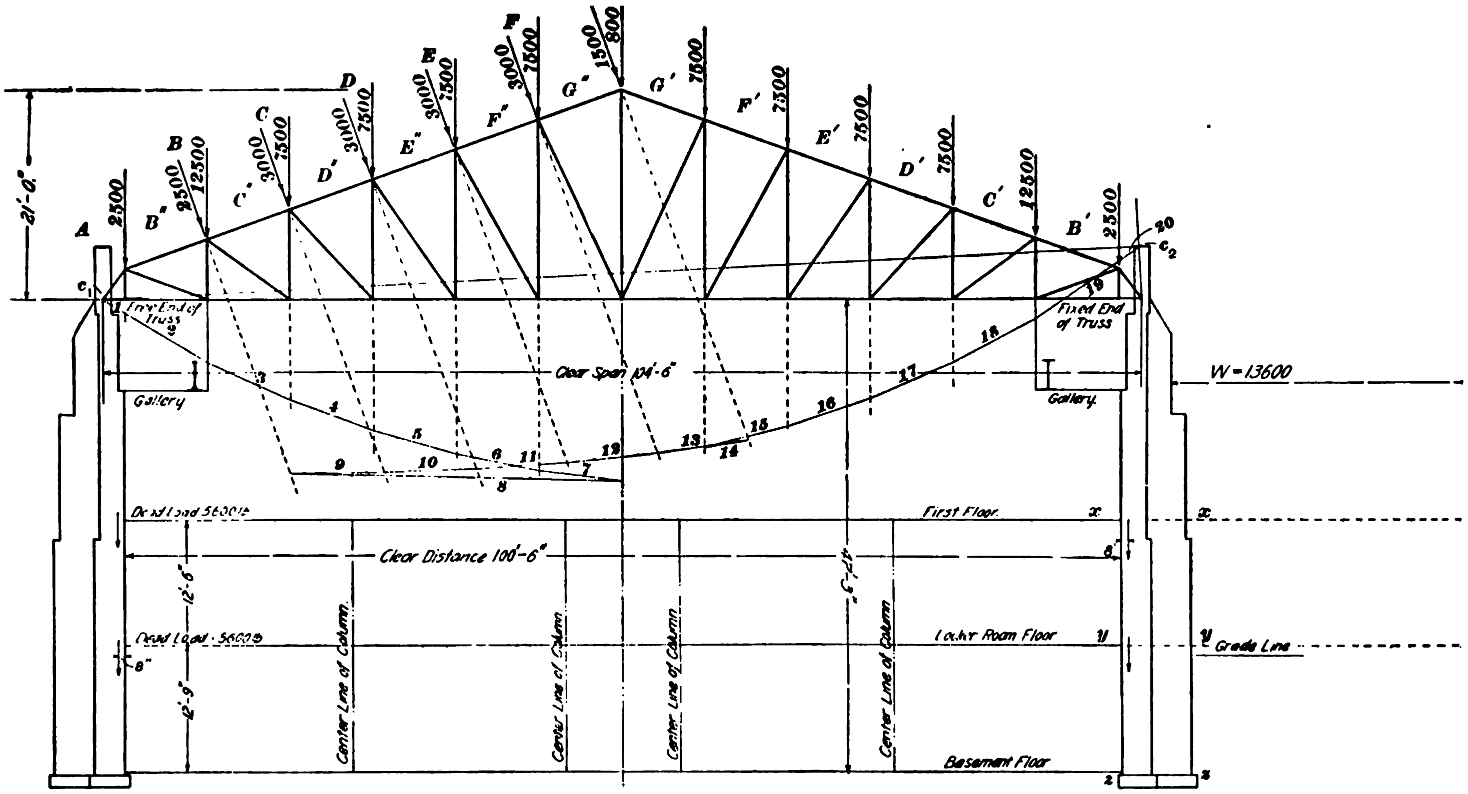
A
B
C
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2500
2500
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7500
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F"
G"
G'
F'
E'
D'
C'
B'
c_1
c_2
20
19
Free End of Truss
Fixed End of Truss
Gallery
Clear Span 104'-6"
W=13600
21'-0"
Dead Load 56001
Dead Load 56005
First Floor
Locker Room Floor
Basement Floor
Grade Line
Clear Distance 100'-6"
Center Line of Column
47'-3"
12'-6"
12'-9"
8"
x
y
z

(a)

a

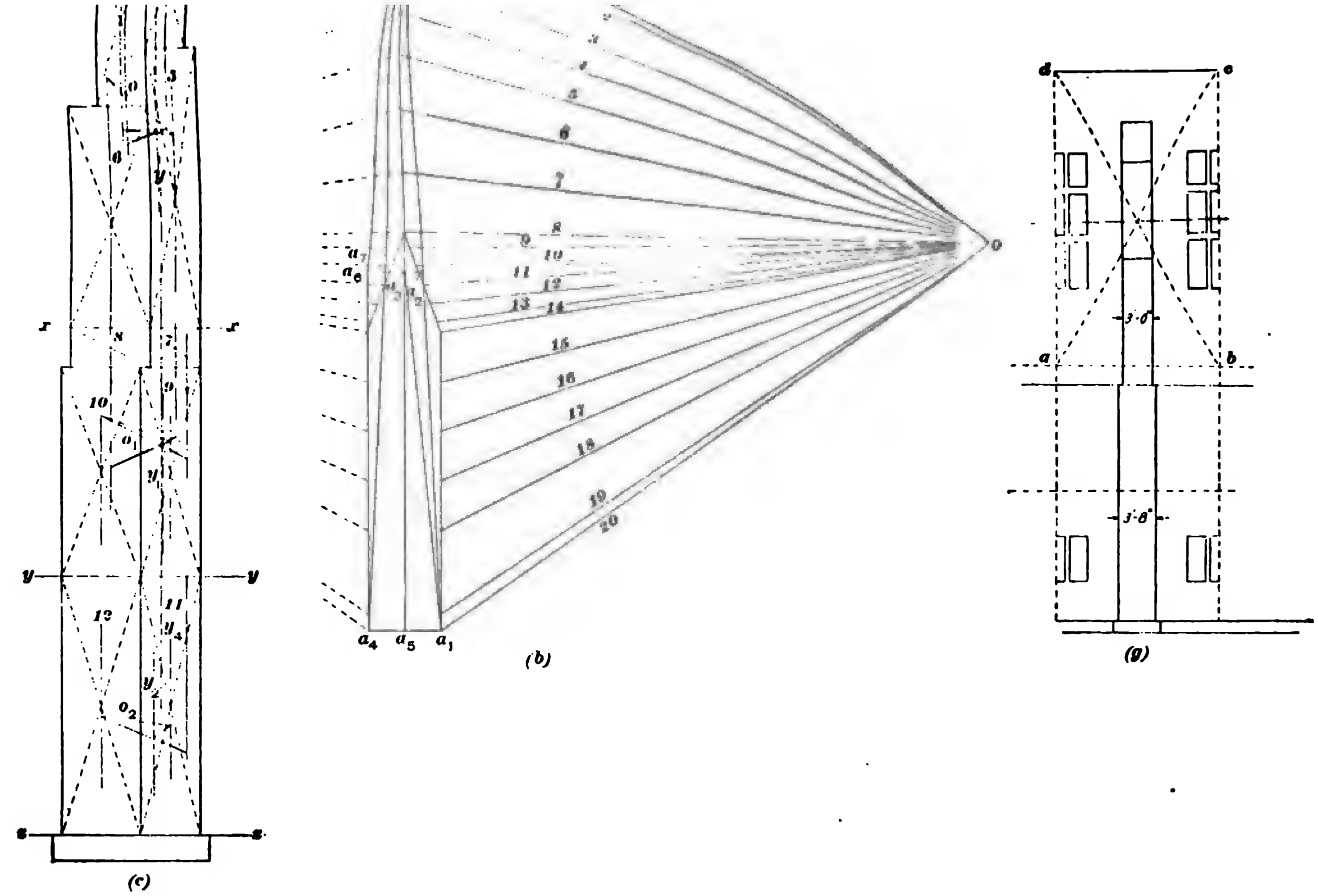

Fig. 47

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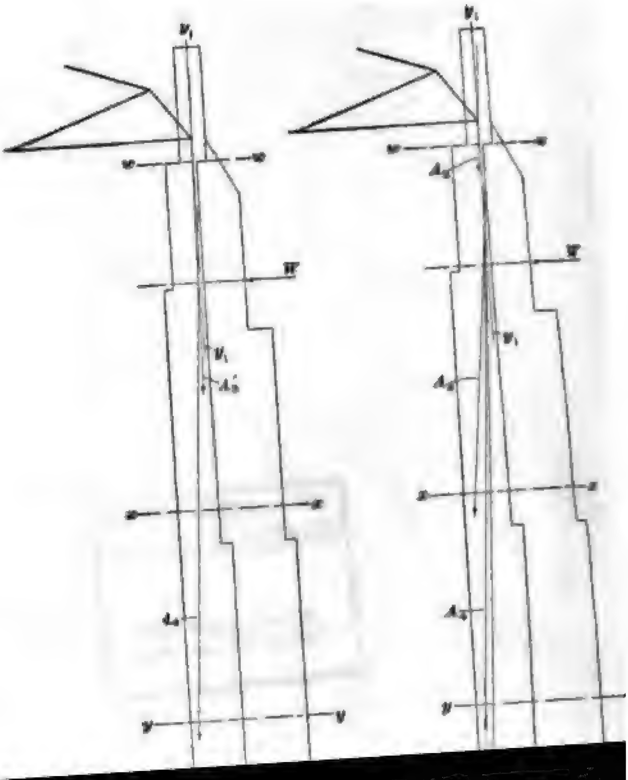
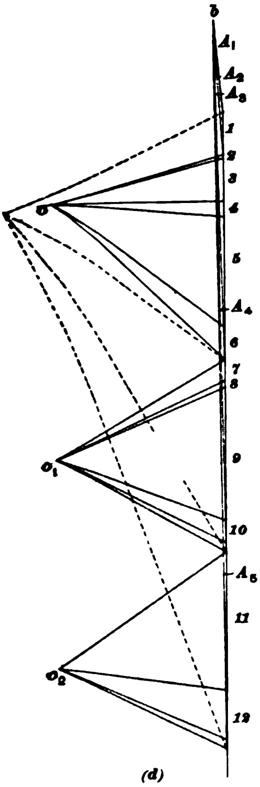


W
A_1
A_2
A_3
A_4
A_5



b
A_1
A_2
A_3
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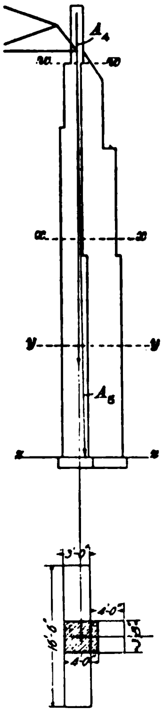
5
A_4
6
7
8
9
10
A_5
11
12
O
O_1
O_2

(d)



A_4
w
w
x
x
y
y
A_5
z
z
5'-0"
4'-0"
16'-6"
4'-0"

(e)

§ 18

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TILDEN FOUNDATIONS.

with the last line of action of the wind pressure, the line *14* is drawn parallel with that line in the force polygon until it intersects the first vertical load to the right of the apex load, and from this last intersection, *15*, *16*, *17*, *18*, and *19* are drawn parallel with the rays in the force polygon, while *20* is drawn coincident in direction with the last ray in the force polygon until it intersects an oblique line that has been drawn through the right-hand heel point of the truss in a direction parallel with $a a_1$ in the force polygon. The intersection of the line *20* with a line drawn coincident in direction with the reactions of the truss, which would exist provided that the ends were fixed, will locate the point c_2. The points c_1 and c_2, the terminals of the equilibrium polygon, may now be connected and a line drawn in the force polygon from the pole o parallel to it. This line is shown dotted, and by intersecting the line $a a_1$ at the point a_2, the amounts of the right-hand and left-hand reactions at the ends of the truss, provided that they are fixed, will be determined.

Since, however, the truss is fixed at but one end, while the other is on rollers, the reactions do not act in the same direction at both ends. From the fact that a roller bearing can supply no horizontal resistance, except the little due to friction, the reaction at the left-hand heel of the truss is vertical and must necessarily be the vertical component of the oblique reaction at the end of the truss, provided that the truss was fixed at the ends. In order to find the amount of this vertical reaction and knowing its amount and direction, to find the other reaction as well, which is likewise changed by the conditions of the end, all that is necessary is to extend from a_2 a horizontal line and drop from a a vertical line, thus obtaining a_3, when $a a_3$ will give the amount of the vertical reaction at the left-hand end, and $a_1 a_3$ the direction and amount of the oblique reaction at the right-hand end of the truss.

The reactions that have just been found are the set required to maintain the truss in equilibrium when the wind blows from the left, but when the wind pressure is against the right-hand slope of the roof, the reaction at this end of the truss will be

changed in direction and in amount. It is evident that the direction of the reactions with the wind from the right, provided that the ends of the truss are fixed, will be that of $a\,a_4$ in the force polygon. This line has the same obliquity as $a\,a_1$, and is drawn by laying off the distance $a_4\,a_5$ equal to $a_5\,a_1$. It is evident also that the system of loading with the wind from the right is just the reverse of the system of loading when the wind acts from the left. In consequence of this similarity, the equilibrium polygon will be of the same form and the line $c_1\,c_2$ will slope the same amount in the opposite direction. The divisions of the line $a\,a_4$ will be the same as the divisions of the line $a\,a_1$, and the force polygon for the wind on the right will be the same as in Fig. 47 (*b*) reversed, as suggested by the dotted lines.

If the reactions for the truss fixed at both ends with the wind from the right are represented by $a_4\,a_7$ and $a_7\,a$, then $a_4\,a_6$ is the vertical component of $a_4\,a_7$ and is consequently the reaction at the roller end of the truss, while $a_6\,a$ is the oblique resultant of the remaining vertical and the entire horizontal component of the reaction at the fixed end of the truss. Hence, two forces $a_6\,a$ and $a_1\,a_3$ are the reactions at the right-hand end of the truss, whose action on the wall it is desired to find.

55. In the investigation of the stability of the right-hand wall shown in Fig. 47 (*a*) from the overturning tendency of the wind blowing from either the right or left, it is safest to find the position of the final resultant force or action at each of the bed joints $x\,x$, $y\,y$, and $z\,z$, as well as on the foundation soil.

The two forces acting on the right-hand wall when the wind blows from the left are the oblique action at the end of the truss and the vertical action due to the weight of the wall. The same wall, however, when the wind blows from the right, is subjected to the action of three forces; viz., the oblique action from the end of the truss, the horizontal pressure of the wind against the vertical face of the wall, and the weight of the wall acting vertically. The first step in

analyzing the resultant of these forces is to find the center of gravity of each portion of the wall that it is desired to investigate.

In Fig. 47 (*c*) is shown an enlarged section of the wall drawn to scale. It is desired to first find the position of the center of gravity of the portion *1*, then the position of the center of gravity of the section of the wall made up of the portions *2*, *3*, *4*, *5*, and *6*, also the center of gravity of the section of the wall made up of the portions *7*, *8*, *9*, and *10*, and finally the center of gravity of the section of the wall made up of the portions *11* and *12*. In determining these centers of gravity, the actual weights must be calculated, from the fact that the section of the wall is not uniform throughout its length, for the abutments are included in the section of the wall under consideration and only project from a portion of it. Either the graphical or the mathematical method may be employed for this purpose; in this instance, as the solution is entirely graphical, the graphical method is employed. The center of gravity of the portion *1*, or of any rectangular portion in the section, is best located by drawing the diagonals, as shown by the dotted lines. When the center of gravity for each portion of the wall has been obtained in this manner, the vertical load line shown in Fig. 47 (*d*) should be laid out, each division of the load line being made equal, by scale, to the weight of the elementary portions of the wall.

In this diagram, the divisions of the load line are lettered similarly to the portions in view (*c*). The weights due to the dead loads from the floor girders opposite the buttresses are represented in this load line, and undoubtedly influence the results of the discussion. In reference to the consideration of the floor loads and their action on the wall, it is necessary to say that some engineers take account of the live load as well as the dead load. This, however, is a matter of judgment, and it is a question whether such an indeterminate and intermittent load may be considered in analyzing the stability of a buttressed wall. The dead loads from the first and locker-room floors are shown in direction

and position in Fig. 47 (*c*), and are likewise shown in the divisions of the vertical load line. When the load line has been laid off in this manner, the poles o, o_1, o_2 may be chosen and the rays drawn, as shown. In this manner three force polygons are described, and from these the equilibrium polygons o, o_1, o_2, Fig. 47 (*c*), may be drawn in the usual manner. The method of drawing the equilibrium polygon has been described in this discussion, as well as explained at length in *Graphical Analysis of Stresses*, Part 1, while an application of the principle of finding the center of gravity has been described in *Properties of Sections*. These several equilibrium polygons will give the lines y, y_1, and y_2, which are the locations of the planes passing through the centers of gravity of the three parts of the wall section shown in Fig. 47 (*c*). As these lines lie close together it is evident that the center of gravity of the entire wall section lies between them or between y_2 and y_4, which latter passes through the center of gravity of portion *1*.

By drawing a new force polygon, which successively includes each section of the wall and which is shown by the dotted lines in view (*d*), the location of the center of gravity of the entire section of the wall above such planes as $w w$, $x x$, $y y$, and $z z$ may be determined. In order to avoid confusion, however, the equilibrium polygons necessary for determining the centers of gravity for all of the wall above each of these planes will not be drawn. They will be found, however, to lie so close together in this case as to nearly coincide, and can therefore be represented by a single line. Assuming, therefore, that the center of gravity of the portion *1* and for the section combining the portions *1*, *2*, *3*, *4*, *5*, and *6*, also for the sections combining *1*, *2*, *3*, *4*, *5*, *6*, *7*, *8*, *9*, and *10*, as well as the sections including all of the portions of the wall have been found, and located in (*e*) and (*f*) by a vertical line which is designated as $y_1 y_1$ in (*f*), the analysis and the determination of the direction of the resultant force from the action of the several forces of the wall may be found. It is necessary to redraw the section of the wall to a larger scale, as shown in view (*e*), and to lay off,

in the force polygon (*d*), from the upper terminal of the load line, a line A_1 that is parallel with the reaction $a_1 a_2$ in the force polygon (*b*). This line is laid off, to scale, equal to the amount of this reaction; in this manner the point *b* is located. If from *b* lines are drawn to the upper terminal of the line representing the weight of the portion *2*, to the upper terminal of the line representing the weight of the portion 7, to the upper terminal of *11* and to the end of the vertical load line, and designated as A_2, A_3, A_4, A_5, they will represent the direction and amount of the resultant actions occurring in the wall from the oblique reaction from the truss and the weight of the several sections of the wall above the planes *w w*, *x x*, *y y*, and *z z*. To determine the reaction of these resultant actions in the wall it would ordinarily be necessary to draw, in Fig. 47 (*e*), lines representing A_2, A_3, etc. through the intersection of the oblique reaction from the truss and the vertical line through the center of gravity of the section of the wall above the respective plane; but in this instance, since all the lines passing through the centers of gravity of the portions above the several planes are assumed to pass through the heel of the truss, only lines A_4 and A_5 have been drawn. It will be noticed that in this instance the line A_5 or final action falls close to the outside edge of the wall, showing that the wall would be in unstable equilibrium were it not for the reenforcement of the abutment.

56. It is now necessary to consider the action on the right-hand wall with the wind blowing from the right. This condition is somewhat more complex than the previous one, from the fact that there is a horizontal action of the wind blowing against the surface of the wall, as designated by *W* in Fig. 47 (*a*). In determining this wind pressure, again the judgment of the designer is exercised; some would take the entire wind pressure on the exposed surface and consider it applied at the center of the height of that area. In this instance, however, from the fact that the floors lend additional support to the wall and because the action of the

wind near grade is materially nullified, only the wind pressure on that portion of the wall included in *a b c d* in Fig. 47 (*g*) is considered, and its center of effort is assumed to be applied at the center of the area determined by drawing the diagram. In order to avoid confusion of lines, the right-hand wall is again drawn in section, view (*f*), and the stress diagram is laid off as in view (*h*).

An inspection of the section shows that the grade line is high on this side of the building, so that if the soil were of a granular or loose nature, the horizontal action from the thrust of the earth, besides the action of the wind, would have to be introduced into the analysis. In this instance, however, it is assumed that it is natural earth that has been excavated and that it is very stable, so that the little pressure on the wall is disregarded. In determining the position of the several lines marked A_2, A_3, A_4, A_5 in view (*f*), it is necessary to lay off the load line shown in view (*h*), which has the same divisions, since it represents the weight of the same wall as the load line shown in view (*d*). The reaction A_1, however, has a different direction, for it must be drawn parallel with $a\,a_6$ in the force polygon (*b*), and must be made equal in amount, by scale, to this force.

The horizontal action of the wind is not considered as influencing the line of action of the resultant that passes through the plane $w\,w$, so that the action A_2 is the line of action of the resultant that intersects this plane.

57. In determining the resultant actions at the planes $x\,x$, $y\,y$, and $z\,z$, the horizontal wind pressure should be introduced, and it is necessary to lay off the force W in the load diagram (*h*) equal to the horizontal wind pressure. The lines A_3, A_4, and A_5 representing the resultant actions through the wall sections in Fig. 47 (*f*), are drawn from the intersection of the horizontal wind action with the resultant action of the vertical force representing the weight of the masonry above each section and the oblique action from the end of the truss.

The resultant that locates the intersection on the line of

action of the wind pressure is drawn from the intersection of the oblique action at the end of the truss and the vertical line passing through the center of gravity of the section of the wall above the plane under analysis.

The method of drawing the several lines of action just explained is more clearly demonstrated in view (*i*), where only the line of action intersecting the plane yy is illustrated, and consequently a confusion of lines is avoided. In this illustration the vertical line y_1y_1 is first drawn through the center of gravity of all the wall and buttress section above the plane yy, the location of this line having been found by the method that has been explained.

The line A'_3 in (*i*) is drawn parallel with A'_3 in (*h*) which is the resultant of the oblique reaction of the truss and the weight of the portion of the wall above yy, and is drawn through the point of intersection of the oblique reaction with the vertical line y_1y_1, which in this case is through the heel of the truss.

To locate the action A_4, with reference to the wall section, it is necessary to draw from the intersection of line A'_3, and the line of action of the wind pressure W, extended, a line parallel with A_4 in the force polygon (*h*). Where this line intersects the line yy is the point at which the resultant of the combined actions of the wind loads on the truss and wall and the weight of the wall section is concentrated. The direction of this resultant is given by the line A_4, which represents it, while its amount may be found by scaling A_4 in view (*h*).

The several actions have been described through the wall section shown in Fig. 47 (*f*) in the manner described in connection with the section in (*i*), so that the diagram (*f*) gives the positions of the several actions at the different beds or planes, with the wind from the right. Having found the lines of action of the several resultants and the location of their intersection with the several planes of the wall section, the bearing area adjacent to these intersections must be investigated to ascertain whether the safe crushing resistance of the material is exceeded. It must likewise

be determined whether the footings are so designed and the area of them is so disposed that there is no likelihood of the soil being compressed to such an extent as to endanger the stability of the wall. The several principles governing the investigation of the action of the resultants on the several beds of the masonry are explained in *Statics of Masonry*, Part 1, under the heading Bearing Blocks. It will not be necessary to consider at length the action of each of the resultants; it is sufficient for purposes of demonstration to analyze, say, the action of the resultants A_4 on the plane yy in views (*e*) and (*f*). The student is, however, reminded that in working out the actual design of such a structure, each section of each wall should be carefully studied.

58. In Fig. 47 (*e*), the plan of the wall section on the plane yy is shown and the point of application of the resultant A_4 is designated thereon. The amount of the resultant A_4, as measured from the force or load polygon (*d*), is found to equal about 298,000 pounds. Referring to the plan of the wall in view (*e*), it is evident that there is an area symmetrically disposed with reference to the location of the point of application of the resultant A_4, equal to the shaded portion, or 14.66 square feet. It will evidently be on the safe side to consider the resultant A_4 as being concentrated on this area, so that the pressure per square foot is $298{,}000 \div 14.66 = 20{,}327$ pounds. This imposed pressure of 20,327 pounds per square foot is equal to 141 pounds per square inch, which is within the safe bearing resistance of squared rubble laid in cement mortar, of which the wall is constructed.

Referring to Fig. 47 (*f*), the plan shows the area adjacent to the point of application of the resultant A_4 with the wind from the right. The amount of this resultant from the force diagram (*h*) is equal, by scale, to about 295,000 pounds, and as the area shown cross-sectioned is 16.5 square feet, the unit crushing stress on the masonry at the plane or bed yy is equal to 124 pounds, which is less than the safe unit bearing strength of the masonry. In these investigations, considerable refinement could have been employed in the analysis by

considering the adhesive resistance of the mortar in the joint and calculating the resistance of the entire wall bed at the different planes. For all practical purposes it is sufficient, however, in determining the stability of such a buttressed wall, to observe whether there is sufficient bearing area adjacent to the point of application of the resultant. In other words, if the wall has sufficient area in the shaded portions of the plans in (*e*) and (*f*), considering the remainder of the bed as scraped away, it is evidently stable; the other element, such as the adhesion of the mortar and the partial distributing of the load over a greater area, tends to increase the factor of safety. It should also be remembered that the conditions of the assumed example are the severest that can probably occur, and that even when they exist the duration of the intense action of the wind pressure is limited.

When investigating the pressure on the soil beneath the footings, the principles applied to bearing blocks and foundations may be employed to determine the maximum unit pressure on the soil. In this example the footings are ample, for the soil is of compact gravel and sand capable of sustaining from 6 to 8 tons per square foot without appreciable settlement.

59. In the construction of all buttressed walls where the buttress is an important structural element, the greatest care must be exercised to secure a thorough bond between the buttress and the wall. The importance of this is evident in such a problem as the one described, in which the entire stability of the wall depends on a unity of action between the wall and buttress.

STATICS OF MASONRY

(PART 3)

ARCHES, VAULTS, AND DOMES

ARCHES

HISTORICAL SKETCH

1. The most ancient form of **arch** known is found at Gizeh, Egypt, where it serves as the ceiling of the chamber in the great pyramid of Chephren. It consists of two abutting stones *a, a*, Fig. 1, over which are laid two layers of stone *b, b* resting against each other and supporting the enormous load of masonry above. This cannot, strictly speaking, be called an arch, since the stones are subjected to a bending moment, and serve as rafters rather than voussoirs.

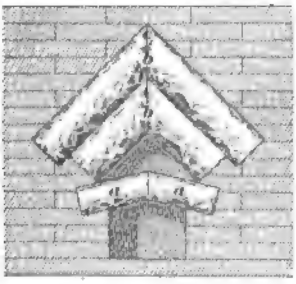

FIG. 1

In Fig. 2 are shown the plan and cross-section of the so-called Treasury of Atreus, at Mycenæ. This is of Greek origin, and is one of the earliest known arched buildings having a curved soffit. It is an underground vault consisting of a series of circular corbels so arranged that the construction might be termed a pointed dome. The vertical forces are taken up in exactly the same way as in a simple corbel, and equilibrium is readily maintained, since the horizontal forces due to the tendency to rotate about the lower courses are taken up at the peak of the vault by all the corbel

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stones coming together and reacting against one another. At the base the horizontal forces are resisted by friction, as well as by the load of earth surrounding the whole vault; thus a practically uniform vertical pressure over the whole mass is created.

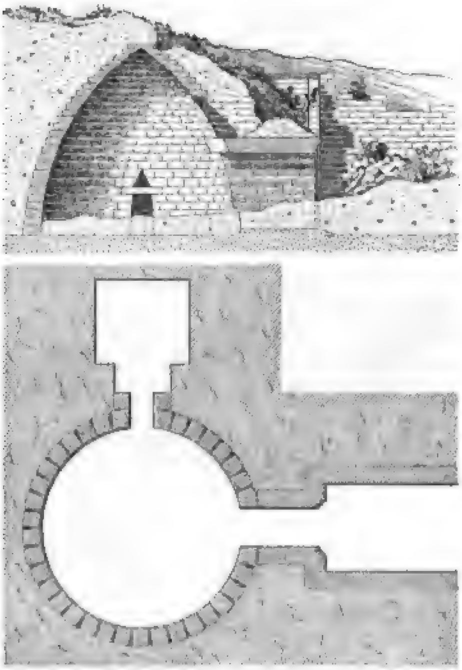


FIG. 2

So far as is known, the Etruscans were the first to employ true arches, or arches made up of voussoirs cut with radiating joints and fitted so as to react against one another. The Roman sewers constructed during the reign of the Tarquins in the early part of the 6th century B. C. are examples of this period.

INTRODUCTION

2. There are in use many methods of proportioning the various parts of arches, but none of them being entirely free from error, it is practically impossible to make a purely

scientific analysis, especially since so many unknown factors must receive consideration.

An elaborate discussion of the arch is not contemplated in this Section, although the student will be given sufficient data to enable him to proportion the various parts according to the best practice of the day.

DEFINITIONS

3. In order to gain a better understanding of this subject, the following definitions of terms used in connection with arches are given; they may be readily understood by reference to Fig. 3.

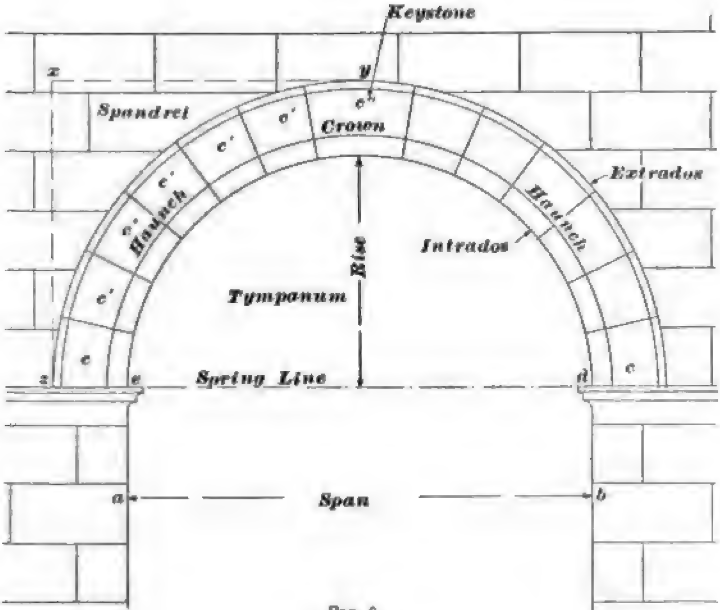


FIG. 3

Span.—The distance between the abutments, as shown at ab, Fig. 3.

Springers.—The lower voussoirs or bottom stones of an arch that lie immediately on the imposts, as at c, c.

Spring Line.—A line drawn through the points where the arch intersects the abutments or where the vertical supports

of the arch terminate and the curve begins, as shown at $e\,d$, Fig. 3.

Intrados.—The lower concave surface of the arch, formed by the under sides of the assembled voussoirs, although considered by some authorities to be the concave line of intersection of the soffit with a vertical plane perpendicular to the axis of the arch.

Soffit.—The lower surface of the arch.

Extrados.—The upper convex surface of the arch formed by the outer sides of the voussoirs; also, considered by some as the convex curve in the same plane as the intrados when the latter is considered as a line.

Rise.—The perpendicular distance from the spring line to the highest point of the intrados.

Arch Ring.—The arch itself contained between the intrados and the extrados.

Voussoirs or Ring Stones.—The stones c, c', c'' making up the arch ring.

Keystone.—The central voussoir c'' of the arch.

Skew Backs.—The two end stones c, c from which an arch springs.

Crown.—The highest portion of the arch.

Haunches.—The portion of the arch included between the crown and the skew backs.

Tympanum.—The space between the spring line and the intrados.

Spandrel.—The triangular wall space included between the extrados, a horizontal line drawn through the apex of the arch, and a vertical line drawn through the extremities of its extrados. The spandrel is shown at $z\,x\,y$, Fig. 3.

Spandrel Filling.—The masonry filling the spandrel.

FORMS OF ARCHES

4. Arches derive their names from the curve of the intrados, as *semicircular*, *segmental*, *flat*, *pointed*, *elliptical*, *horseshoe*, etc. By far the most common form is the segmental in its true form; most of these, however, have a special form of skew back.

FAILURE

5. The **failure** of arches may be caused: First, by the crown falling and the haunches rising, as shown in Fig. 4 (*a*); second, by the crown rising and the haunches falling, as shown in Fig. 4 (*b*); third, by the crushing of the voussoirs due to overloading; fourth, by the spreading of the abutments, as shown in Fig. 4 (*c*).

The first is liable to take place when the load acting on the crown is much in excess of that which the haunches are called on to support, while the second occurs when these conditions are reversed, that is, when the haunches are more heavily loaded than the crown, also in the case of a Gothic arch whose crown is too sharply pointed. The third takes place only in cases where the loads are excessive; but the fourth must be most carefully taken into consideration, since in this way the greatest number of arches fail.

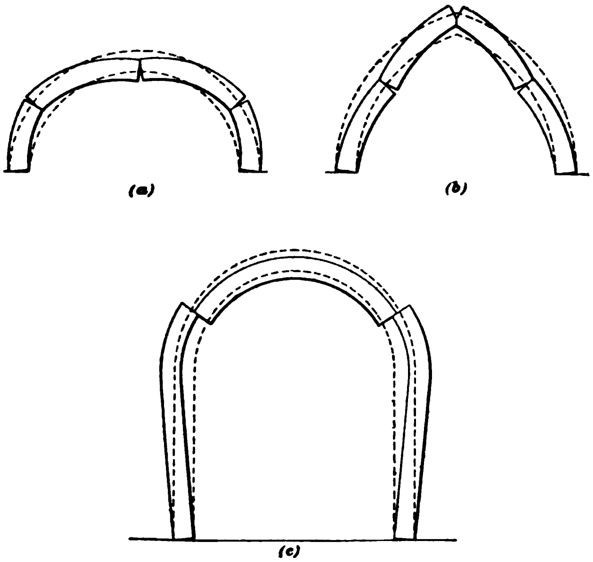


FIG. 4

RULES DERIVED FROM PRACTICE

6. In designing, the engineer is influenced largely by the general appearance of the arch, and this, as far as is practical, must conform to the theoretical conditions of safety, which are in reality but an approximation of the actual conditions. He usually has given the span and the kind of masonry to be used in both arch and spandrel filling, and must then assume the remaining proportions according to the general engineering practice. In cases when the span of the arch is small, as in window openings, when there is a backing of solid masonry, all that is required is to obtain the general outline by making the joints radial and the arch ring sufficiently deep. The following rules are of importance in proportioning arches.

7. To Determine the Proper Rise for a Given Arch.—The proper rise for a given arch may be determined by applying one of the following rules:

Rule I.—*For rubble masonry, the rise should not be less than one-sixth the span, or a rise of 2 inches per foot of span.*

Rule II.—*For common hard brick set in lime mortar, the rise should not be less than one-eighth the span, or a rise of $1\frac{1}{2}$ inches per foot of span.*

Rule III.—*For very hard brick set in cement mortar, the rise should not be less than one-twelfth the span, or a rise of 1 inch per foot of span.*

If Pompeian brick is used the rise need not be so great, since in this case the length is greater than in common brick. For cut stone used in arches of short span, say 4 or 5 feet, the rise may be zero, while for longer arches the rise may be made one-twelfth the span.

8. To Determine the Radius of a Given Arch. After the rise and span have been determined, the radius may be found by the following rule, in which all dimensions are taken in feet:

Rule.—*The radius of the arch is equal to the quotient obtained by dividing the sum of the square of the span and four times the square of the rise by eight times the rise.*

This rule may be expressed by the following formula:

$$R = \frac{S^2 + 4H^2}{8H} \qquad (1)$$

in which H = rise, in feet;
S = span, in feet;
R = radius of curvature of intrados, in feet.

EXAMPLE.—The rise of an arch is 6 inches and the span 3 feet; what is the radius?

SOLUTION.—In this case, $S = 3$ ft. and $H = .5$ ft.; hence,

$$R = \frac{3^2 + 4 \times .5^2}{8 \times .5} = \frac{9+1}{4} = 2.5 \text{ ft. Ans.}$$

9. Determining Depth of Keystone.—The depth of the keystone may be determined by applying the following rule, in which all dimensions are taken in feet:

Rule.—*Find the square root of the sum of the radius and half the span of the arch; divide the result by 4 and to the quotient add 2.*

This rule may be expressed by the following formula:

$$D = \frac{\sqrt{R + \frac{1}{2}S}}{4} + .2 \qquad (2)$$

in which D = depth of arch at crown, in feet;
R = radius of curvature of intrados, in feet;
S = span, in feet.

This formula holds good in first-class cut-stone arches; but in figuring for second-class cut-stone work, one-eighth of the depth thus found should be added, while for brick and fair rubble, one-third should be added.

EXAMPLE.—Assuming that the arch mentioned in Art. 8 is first-class cut stone, what is the required depth of the keystone?

SOLUTION.—Substituting values in formula 2, $D = \frac{\sqrt{2.5 + \frac{3}{2}}}{4} + .2$, or $D = \frac{2}{4} + .2 = .7$ ft., or 8.4 in. Ans.

EXAMPLES FOR PRACTICE

1. How deep must the keystone of a brick arch be made if the span is 10 feet and the radius 7 feet 3 inches? Ans. 1 ft. 5.2 in.

2. What will be the radius of curvature of the intrados of an arch whose span is 30 feet and rise 7 feet? Ans. 19 ft. $6\frac{6}{7}$ in.

3. It is required to build a first-class cut-stone arch having a span of 24 feet. Using the foregoing rules, what would be the rise, radius, and depth of keystone?

Ans. $\begin{cases} \text{Rise} = 2 \text{ ft.} \\ \text{Radius} = 37 \text{ ft.} \\ \text{Depth of keystone} = 1 \text{ ft. } 11.64 \text{ in.} \end{cases}$

JOINTS

10. In most arches, such as those formed by certain combinations of curves, and in the ordinary circular arches, the **joints** are radial and hence perpendicular to the curve of the intrados. A noteworthy exception to this rule is the lancet arch, whose joints are not radial, a condition that is

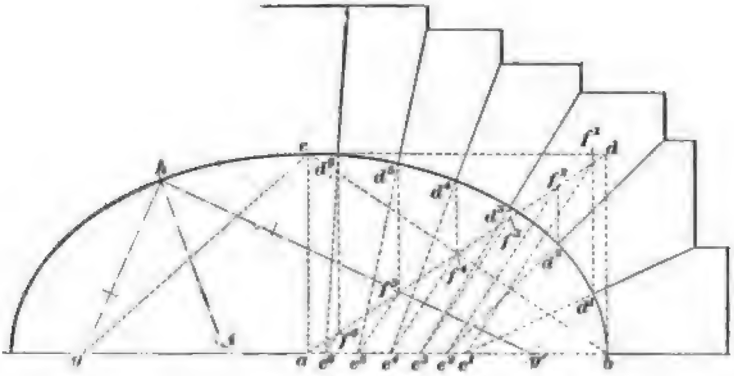


Fig. 5

often detrimental to the arch, since the angle between the line of pressure and the joint is increased, thereby producing a greater tendency to shear.

11. Referring to Fig. 5, d^1, d^2, d^3, d^4, d^5, and d^6 are points through which it is desired to draw joints. Tangents to the ellipse are drawn at the points b and c, intersecting at some point d. Now $a\,d$ and $b\,c$ are drawn, and from d^1, d^2, d^3, etc. lines are drawn parallel to $a\,c$; these lines will intersect $a\,d$ at certain points f^1, f^2, f^3, etc., from which are drawn lines

perpendicular to bc, intersecting ab at e^1, e^2, e^3, etc. Finally, lines e^1d^1, e^2d^2, e^3d^3, etc. are drawn; these lines will give the direction of the joints at d^1, d^2, d^3, etc.

A simpler method of finding the direction of the joints is as follows: Find the foci of the ellipse by striking arcs from c with ab as a radius, cutting the major axis at g and g'. Let h be the point where the direction of the joint is to be found. Draw gh and $g'h$, and bisect the angle ghg', as at hi; then hi is the direction of the joint at h.

LOADS

12. Having determined the proportions of the arch, it should be accurately drawn to the largest scale that may be conveniently used. After this is done, the various **loads** acting on the arch are calculated. Since these vary, it is largely a matter of judgment as to what shall be assumed.

13. Arches supporting earth, as under railroad embankments, etc., are (according to Rankin) subjected to a vertical pressure equal to the weight of the earth and a maximum horizontal pressure equal to from one-third to three times the pressure due to a liquid whose density is the same as that of the earth, and pressing on the vertical projection of the back of the voussoir. The true pressure is somewhere between these extremes, and varies as the intensity with which the earth filling is rammed.

14. When an arch supports loads of several kinds, it is customary to make a load diagram similar to the one shown in Fig. 6, which is an illustration of a cut-stone arch supporting a load of loose earth. The arch itself weighs 160 pounds per cubic foot, while the weight of the earth is 100 pounds per cubic foot. If the earth is filled in 6 feet above the crown of the arch, the height of cut-stone masonry above the arch that will be equivalent in weight to the earth filling above the crown will be $\frac{6 \times 100}{160} = 3.75$ feet. The corresponding heights for several other points may be found, and laid off to scale on vertical lines, and a line ab drawn

through their upper extremities to represent the equivalent height of cut-stone masonry above the arch. Other loads are reduced to the same standard; the diagram is then drawn and the area of each strip multiplied by 1 foot and the weight of the stone per cubic foot; the result, as in this case, then represents the amount of the load on any voussoir considered.

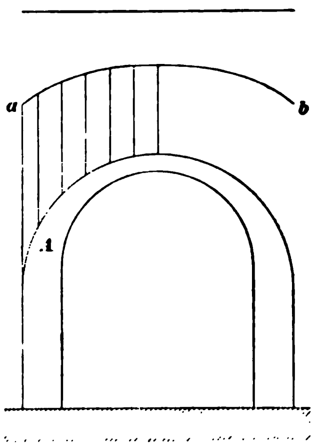
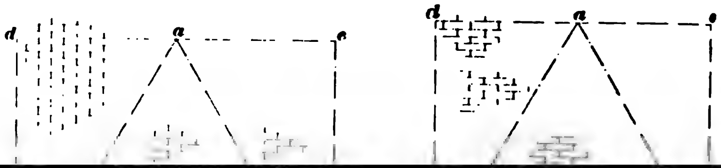


Fig. 6

15. When an arch supports a solid masonry wall, as in Fig. 7, lines *a b* and *a c* are drawn tangent to the arch, if semicircular, as in (*a*), or to the ends of the extrados, if segmental, as in (*b*). These lines should be drawn at an angle of 60° with the horizontal. Through the apex of the triangle *a b c* in (*a*) a horizontal line *d e* is drawn and prolonged until it intersects the two vertical lines *d f* and *e g* drawn through the extremities of the extrados. In the same



on the arch. In support of this theory, it may be said that if the arch were calculated for the triangular load *a b c*, a lintel would usually be considered, and the line of pressure would be outside the arch ring at the haunches, indicating a tendency to rise, as in Fig. 4 (*a*), but it is prevented from rising by the weight of the masonry above the spandrels.

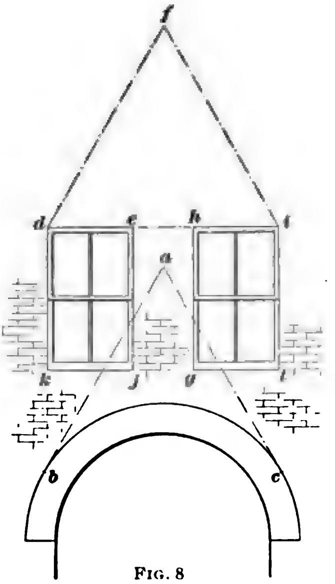


FIG. 8

If the triangle is broken by windows, as shown in Fig. 8, the arch must be figured to carry all the load below the window as well as the concentrated load *j e d f i h g*, which should be considered as acting on the joint *j g*. If the second triangle *d f i* is either partially or entirely broken, the load of the next pier must be considered in the calculation, and although it is poor practice to place windows so that the central pier rests on an arch, it sometimes becomes necessary to arrange them in this manner.

16. Theory of Horizontal Thrust of Masonry. Arches set in a solid masonry wall are in little danger of failure if the abutments are sufficient to resist the thrust. It is claimed by some authorities that there is a horizontal pressure against the back of the arch, which is assumed to be expressed by the formula

$$H = \tfrac{1}{3} w d l \qquad (3)$$

in which w = weight of masonry per cubic foot;
d = distance, in feet, from upper limit of masonry above arch to center of back surface of voussoir;
l = vertical projection of surface, in feet;
H = horizontal pressure on surface.

The values d, l, and H are indicated in Fig. 9, in which cd represents the upper surface of the masonry, ab represents the back of the voussoir, and e, the center of pressure of the horizontal force H. In this formula it is assumed that the horizontal pressure on any voussoir is equal to the least pressure exerted by the earth.

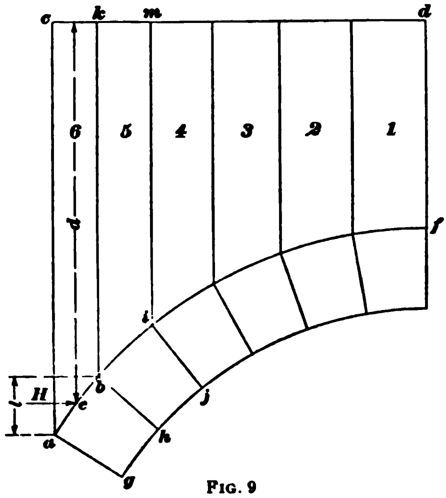


FIG. 9

17. The arch ring is divided into a number of voussoirs, as shown by the lines bh, ij, etc. in Fig. 9, from the outer ends of which vertical lines bk, im, etc. are drawn, dividing the masonry into strips.

For the sake of convenience, all arches are figured as being 1 foot thick, so that the area of the surface, in square feet, also represents the cubical contents of the load. Beginning at the crown, each voussoir with the load it supports is numbered.

18. Center of Gravity.—The center of gravity of each load may be found by the following method: Take any voussoir, $adbc$, Fig. 10, and draw the diagonals ab and cd, which intersect at some point e. Lay off on cd a distance cf equal to ed and draw fb. From the middle point of fb draw ga. Divide the line ga into three equal parts; the center of gravity will be located one-third the length of ga from g, as indicated at i.

In this method, the curved portions ac and db are considered to be straight, as indicated in the figure by the dotted lines. Hence, the center of gravity found is in reality the center of gravity of the trapezoid $adbc$. In the same way

the center of gravity of the load above the voussoir may be found. The two points *j* and *n* being located, a vertical line *n o* is drawn from *n*, and the horizontal distance of the point *j* from *n o* is accurately measured. Then the quotient obtained by dividing the product of this distance and the weight of the voussoir by the sum of the weights of the voussoir *a d b c* and the masonry *h i a c* gives the distance of the center of gravity of the combined weights from the line *n o*.

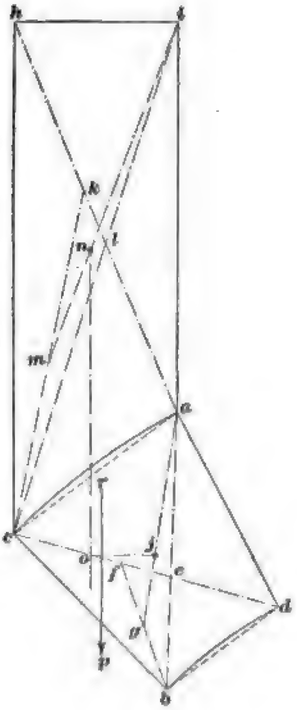


Fig. 10

19. Crown Thrust.—There is a pressure existing at the joints of an arch that is due partly to the weight of the arch and partly to the load that it supports. Many theories have been advanced as to how the line of pressure shall be obtained, but they differ mainly in the determination of the intensity of the **crown thrust** assumed. This thrust may be determined in two ways: First, by the method of moments; and second, by the graphic method.

One-half the arch rests against the other half, producing a horizontal thrust at the crown. The thrust is assumed to act at a point *o*, Fig. 11, one-third the depth of the arch ring from the crown, if no allowance is made for tension; but if the tension is considered, the point *o* is located one-fourth the depth of the arch ring from the crown. This method of making no allowance for tensile stresses in masonry is called the *gravity system*, but where tension is considered, the system of calculating is called the *cohesive system*. In modern work, where good cement mortar is used, it is customary to adopt the cohesive system, and to assume

that the point of application of the crown thrust is one-fourth the depth of the arch ring from the crown. In Fig. 11, two segments of circles *o u* and *v b'* are drawn at one-fourth and three-fourths the depth of the arch ring from the extrados, respectively; these give the limits of the line of pressure.

1. *Method of Moments*.—In order to find the intensity of the crown thrust, the moments of the loads *1, 2, 3, 4*, etc. must be taken about the inner limit of the curve of pressure. For example, the horizontal pressure due to load *1* is equal to

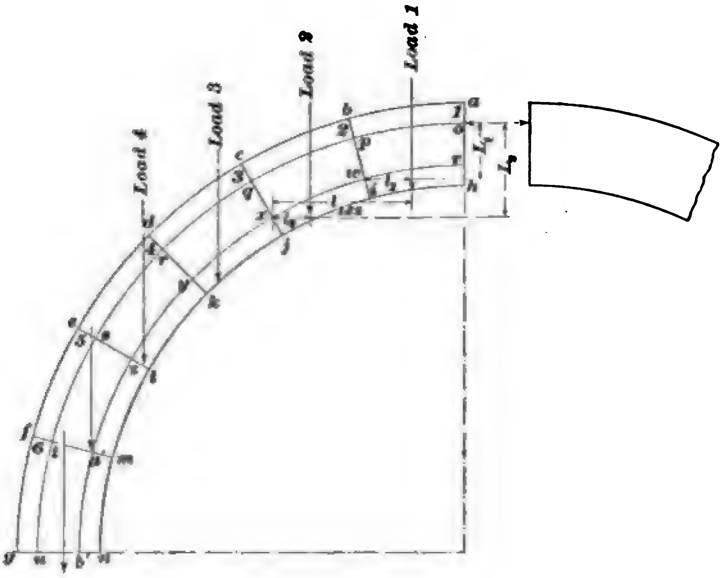


FIG. 11

the weight of load *1* multiplied by l_1 divided by L_1, and similarly, the horizontal thrust due to the loads *1* and *2* is equal to the weight of load *1* multiplied by l_{1a}, and the weight of load *2* multiplied by l_2, and their sum divided by L_2. In segmental arches of less than 110°, the greatest crown thrust will be found at the skew back; but in semicircular arches, the greatest thrust is usually found to be near a line drawn through the center from which the arch is struck, and at an angle of 35° with the horizontal.

2. *Graphic Method.*—In Fig. 12, the center of gravity of each load is found as in Art. 18 and the load line *ag* is

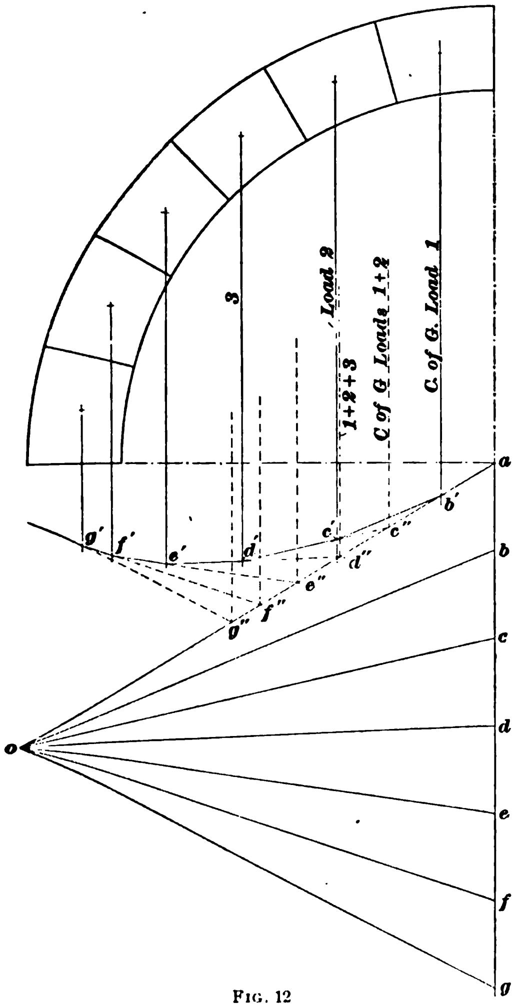


FIG. 12

drawn under the central joint of the arch. On this line, load *1* is laid off from *a* to *b*; load *2*, from *b* to *c*, etc.; from any point *o* without this line, the lines *oa*, *ob*, *oc*, etc. are

drawn. The point o should be selected so that the angles between the lines are nearly equal and at the same time as large as possible, because the greater the angle included between two lines, the more readily can their intersection be located. From the intersection of the line $o\,a$ with the vertical line drawn through the center of gravity of load *1*, $b'\,c'$ is drawn parallel to $o\,b$ and prolonged until it intersects the continuation of the vertical line drawn through the center of gravity of load *2* at c'; similarly, from c', a line $c'\,d'$ is

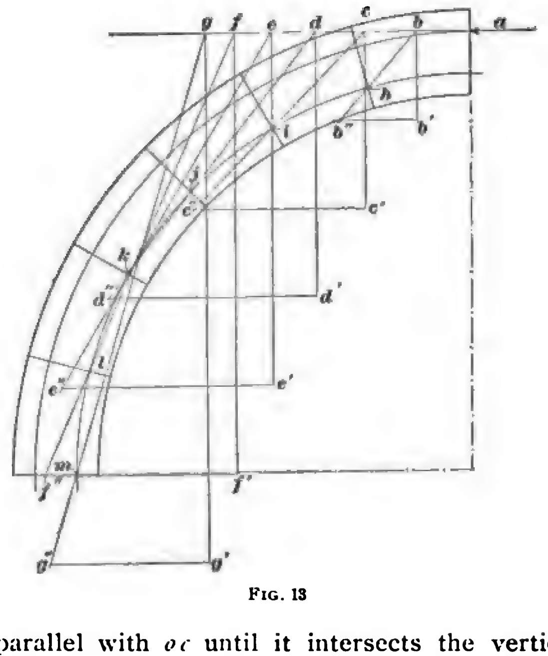


FIG. 13

drawn parallel with $o\,c$ until it intersects the vertical line drawn through the center of gravity of load *3*. Continuing the line $c'\,d'$ until it intersects $o\,a$ at c'', the point of intersection c'' with the line $o\,a$ gives the center of gravity of the combined loads *1* and *2*, and d'', the point of intersection of the line $c'\,d$ prolonged, with $o\,a$, gives the center of gravity of the combined loads *1*, *2*, and *3*. By continuing this method, the center of gravity of all the combined loads may be found.

In order to find the horizontal thrust for these combined loads, the horizontal line ag, Fig. 13, is drawn. From the intersection of ag with the line of action of load *1*, bb'' is drawn through the intersection of the lower limit of the line of pressure with the first joint. To some convenient scale, the distance bb' is laid off on the load line to represent the actual weight of load *1* and $b'b''$ is drawn horizontally until it intersects the slanting line bb''. This line $b'b''$, measured to the same scale as bb', gives the intensity of the horizontal thrust due to load *1*. From c, on the line of pressure of loads *1* and *2*, cc' and cc'' are drawn as before and, on cc', a distance equal to the sum of loads *1* and *2* is laid off. The horizontal line $c'c''$ then represents the horizontal thrust due to loads *1* and *2*.

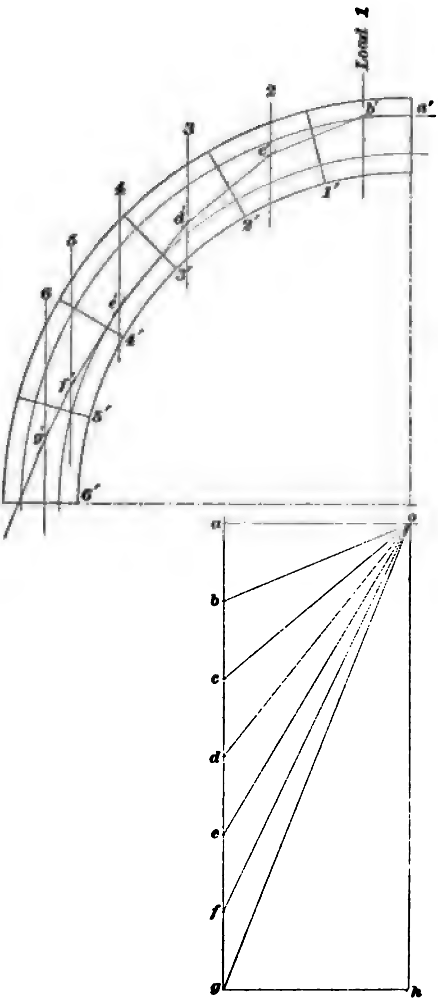


FIG. 14

Proceeding in the same manner at each joint, the horizontal thrust is easily found, and it is seen that the greatest thrust is that shown at $e'e''$, which is due to loads *1*, *2*, *3*, and *4*. This will be the thrust assumed in the calculation.

20. Line of Pressure.—Having calculated the horizontal thrust, the **line of pressure** may be drawn as follows: On the vertical line *ag*, Fig. 14, the loads *1*, *2*, *3*, *4*, etc. are laid off as before. From *a*, the line *a o* is drawn horizontally and on this line a distance *a o* is laid off equal to the maximum horizontal thrust. From *o*, the lines *o b*, *o c*, *o d* are drawn, and at the top of the arch, parallel to *a o* until it intersects the load line *1*, the line *a′ b′* is drawn. From *b′*, a line *b′ c′* is drawn parallel to *o b* until it intersects the load line *2*, and from *c′* a line *c′ d′*, parallel to *o c*, until it intersects the load line *3*. The line of pressure, when completed, should pass through the inner limit of the joint whose horizontal thrust has been considered.

If the arch is safe, the line of pressure should lie within the middle half of the arch throughout its length. The joints having the greatest pressure on them, as at joint *6*, should be calculated by the formula so that the stress on them will not exceed the strength of the material. In order to find the vertical pressure on this joint, the pressure *o g* may be resolved into its vertical and horizontal components; *o h* then represents the vertical pressure and *g h* the sliding component. The angle *g o h* in no case should be greater than 25°.

21. The fact that the line of pressure lies without the middle third does not signify that the arch will fail. Where the strength of the material is great in comparison with the actual pressure, the line of pressure must be wholly without the structure, to cause failure. However, even in some cases where the line of pressure lies wholly within the structure, but the pressure is very great, failure may be caused by the crushing of the parts. This principle is illustrated in Fig. 15. In the experiment, a Pompeian brick *A* is placed on end $4\frac{3}{8}$ inches from the wall. On this, an ordinary brick *B* is placed so that one edge rests on *A*, $6\frac{5}{16}$ inches from the wall. These form practically one-half an arch in which two points, *b* and *d*, of the line of pressure are known, for the horizontal thrust at the crown is at *d* and the line of pressure must pass through the point *b*. The weight of *A* is 5 pounds and of *B*, $5\frac{3}{4}$ pounds. From *d*, the horizontal line *dc* is

drawn until it intersects a vertical line through c, the center of gravity of B. From c, a line representing $5\frac{3}{4}$ pounds is

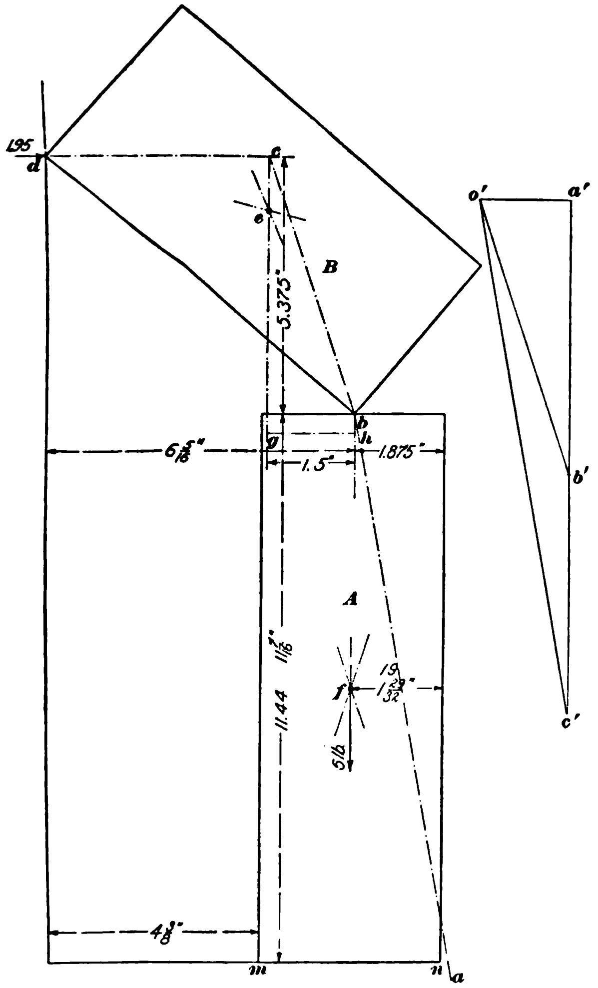


FIG. 15

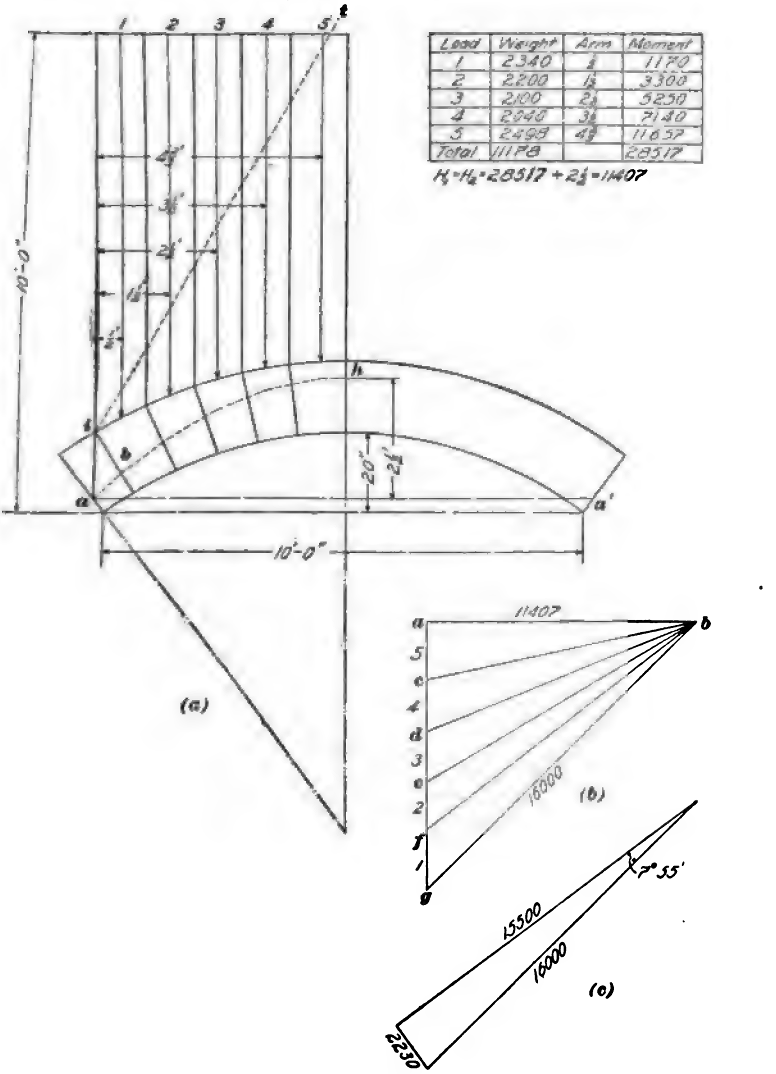
rawn to the point g, and a line gh is drawn horizontally; his line gh represents the horizontal thrust at the point d. Next the force polygon $o' a' c'$ is drawn, by laying off $a' b'$

and $b'c'$ equal to the weights of B and A, and $d'a'$ equal to the horizontal thrust gh. The line of pressure ba is then drawn parallel to $d'c'$ and is found to lie wholly without the base mn. At first sight, this is surprising, but by calculation it is seen that there is a moment about the point n of $(5\frac{3}{4} \times 3.375) + (5 \times 1.9) = 28.91$ inch-pounds, tending to hold the brick A in a vertical position and a moment of $1.95 \times 16.815 = 32.79$ inch-pounds, tending to overturn it; the difference between these moments is 3.88 inch-pounds, in favor of overturning, which clearly shows why the line of pressure lies without the base, at a. In the experiment, the two bricks actually stood, although a little push caused them to overturn. This extra 3.88 inch-pounds is accounted for in probable friction, imperfect edges in bricks, etc.

22. In order to illustrate the principles given, the following example is introduced. In this the analytical method is used to find the horizontal crown thrust.

Example.—What size arch will be required for a 10-foot opening in an 18-inch stone-rubble wall that will be built 10 feet above the spring line of the arch? The stones used for the voussoirs are to be undressed, the same as the wall, set in cement, and are to support the wall above.

Solution.—The first step is to determine the rise and radius. The rise is found to be $\frac{1}{6} \times 10 = 1\frac{2}{3}$ ft. and the radius, from formula **1**, equals $\frac{10^2 + 4 \times (1\frac{2}{3})^2}{8 \times 1\frac{2}{3}} = 8\frac{1}{3}$ ft. Substituting R and S in formula **2**, the depth of the arch ring at the crown should be $D = \frac{\sqrt{8\frac{1}{3} + \frac{10}{2}}}{4} + .2 = 1.113$ ft. But since for rubble the depth must be increased by one-third the actual depth, the keystone must be $1\frac{1}{3} \times 1.113 = 1.484$ ft., or, approximately, 1 ft. 6 in. deep. The rise, radius, span, and depth of keystone having been found, the arch should be laid out to scale, as shown in Fig. 16 (a). Dividing the depth of the keystone by 4, it is found that the point of application h of the horizontal crown thrust is $4\frac{1}{2}$ in. from the top of crown. Similarly, the point a, which is the inner limit of the line of pressure at the skew back, is $4\frac{1}{2}$ in. from the intrados. The portion of wall above the arch is divided into strips, beginning at the point a and working toward the center, and the area of each strip is then calculated. The line aa', shown in the figure drawn through the inner limit of the line of pressure, is called the *theoretical spring line*. In other words, it is the spring line of the theoretical arch. In this case the portion of the backing outside



Load Weight Arm Moment
1 2340 ½ 1170
2 2200 1½ 3300
3 2100 2½ 5250
4 2040 3½ 7140
5 2498 4⅔ 11657
Total 11178 28517
H1=H2=28517 ÷ 2½ = 11407
10'-0"
10'-0"
20"
2½'
4⅔'
3½'
2½'
1½'
½'
(a)
11407
16000
(b)
15500
16000
2230
7°55'
(c)

FIG. 16

the vertical line $a\,i$ is neglected; this gives an extra factor of safety in respect to the line of pressure, as will be seen later. The consideration of this load will reduce the horizontal pressure.

In order to determine whether it is necessary to consider the whole height of the masonry above the arch from the point i, the line $i\,t$ is drawn at an angle of 60° with the horizontal. It is found that this intersects the center line of the arch above the top of the wall. The whole of the masonry wall is therefore taken into consideration. Taking the weight of the wall at 160 lb. per cu. ft., a wall $1\frac{1}{2}$ ft. thick weighs 240 lb. per sq. ft. of wall surface.

	Square Feet	Weight Pounds
Load *1*,	9.75	$9.75 \times 240 = 2{,}340$
Load *2*,	9.16	$9.16 \times 240 = 2{,}200$
Load *3*,	8.75	$8.75 \times 240 = 2{,}100$
Load *4*,	8.50	$8.50 \times 240 = 2{,}040$
Load *5*,	$\frac{8.33 \times 15}{12} = 10.41$	$10.41 \times 240 = 2{,}498$

The moment of each load is given in the tabulation. The moment of the crown thrust around the point a should equal the sum of the moments of the loads *1, 2, 3, 4,* and *5* around the same point; or, in other words, the algebraic sum of the moments should equal zero. The weight of each load multiplied by its arm equals its moment. The sum of these moments is found to be 28,517 foot-pounds, and the vertical distance between the lowest and highest limits of the line of pressure, to be $2\frac{1}{2}$ ft. Hence, the horizontal thrust at the crown is $28{,}517 \div 2\frac{1}{2} = 11{,}407$ lb. This thrust is, of course, balanced by the thrust of the other half of the arch.

Next the force polygon shown in Fig. 16 (b) is drawn by laying off H_1 on the horizontal line $a\,b$ and then laying off, to the same scale, the loads *5, 4, 3, 2,* and *1* from a on the vertical line $a\,g$. These points are connected with b, thus completing the force polygon. Now through the point a, Fig. 16 (a), a line $a\,b$ is drawn parallel to $g\,b$ in (b) until it intersects the load line *1*. This process is continued until the line of pressure is completed. When the last line is drawn horizontally at the crown, its distance from the theoretical spring line should measure $2\frac{1}{2}$ ft. This line of pressure could have been drawn by starting from the point h at the crown of the arch and working down.

By looking at the force polygon, it will be seen that the greatest pressure is represented by the line $b\,g$, which is the pressure on the joint of the skew back and is equal to about 16,000 lb. This line of pressure has the greatest angular departure from the normal, but being only 7° 55′, as shown in Fig. 16 (c), there is no danger of sliding. This joint is the weakest in the arch and if it fulfils all the conditions of equilibrium, the arch will be stable. Resolving the pressure $g\,b$ in (b) into its components parallel and perpendicular to

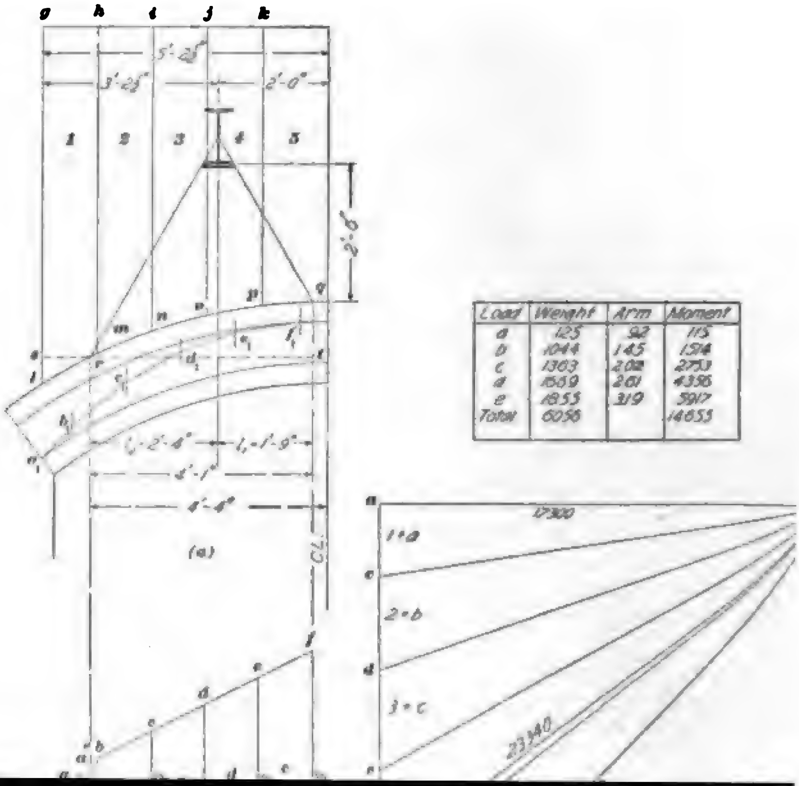
the joint, as shown in (*c*), it is found that the perpendicular pressure on the joint is 15,500 lb. and the tendency to slide is 2,230 lb. Substituting in the formula $k_1 = \frac{2P(2l - 3l_1)}{l^2}$, it is found that the maximum pressure on the joint is $k_1 = \frac{2 \times 15{,}500 \times (2 \times 17 - 3 \times 4)}{17^2} = 2{,}360$ lb. This represents the pressure per inch of width all the way across the arch, and dividing by 17 in., which allows ½ in. on each side for deficiency of joint, the pressure per square inch is 139 lb. As 150 lb. is allowable in this kind of masonry, the arch is safe in regard to compression. The tension in the joint may be calculated in a similar manner and the value of k_2 found. This, when divided by 17, gives a tension on the joint that is within the allowable limit. The dimensions found are therefore satisfactory.

23. Symmetrical Concentrated Loads.—It is often necessary to place a concentrated load, such as an **I** beam, above an arch, or the windows may be so arranged as to necessitate the placing of a pier over an arch. In such cases, most engineers prefer to support this load by a beam set in the masonry. As the principles involved in such a problem should be understood, the following illustration is given to show the methods used.

Take, for instance, an arch similar to that in Fig. 16 (*a*) and supporting an additional weight due to the concentrated loads from two **I** beams symmetrically placed with respect to the center of the arch, as shown in Fig. 17 (*a*). The loads from the beams are assumed to be 6,000 pounds each. When a concentrated load rests on an arch, its bearing should be as large as possible in order to distribute the load over the arch. The beam in the example is 2 feet 6 inches above the crown and 2 feet from the vertical center line of the arch. It has a bearing 7 inches in width, which gives a length of base of 4 feet 1 inch, over which the load is distributed by the corbeling action of the brick. In order to find the maximum and minimum pressures on this bearing, the following formulas are employed:

$$k_1 = (2l - 3l_1)\frac{2P}{l^2}$$

$$k_2 = (2l - 3l_2)\frac{2P}{l^2}$$



Load	Weight	Arm	Moment
a	125	.92	115
b	1044	1.45	1514
c	1363	2.02	2753
d	1669	2.61	4356
e	1855	3.19	5917
Total	6056		14655

[redacted]

In this case, l = 4 feet 1 inch = 4.0833 feet;
l_1 = 1 foot 9 inches = 1.75 feet;
l_2 = 2 feet 4 inches = 2.33 feet;
$2P = 2 \times 6{,}000$ pounds = 12,000 pounds.

For convenience in scaling, these figures are taken in decimal parts of a foot. Then,

$$k_1 = (2 \times 4.0833 - 3 \times 1.75) \times \frac{12{,}000}{4.0833^2} = 2{,}100$$

$$k_2 = (2 \times 4.0833 - 3 \times 2.33) \times \frac{12{,}000}{4.0833^2} = \text{about } 840$$

Referring to Fig. 17 (b), $a'f'$ is laid off to some convenient scale, to represent the length of the bearing, 4 feet 1 inch; perpendicular lines $a'a''$ and $f'f$ are drawn from the ends of the line $a'f'$, and on these the values of k_2 and k_1 are laid off to some convenient scale. A trapezoid is formed by connecting the points a'' and f.

To determine the portion of the weight represented by the trapezoid $a''ff'a'$ that must be added to each masonry load, the lines $b'b$, $c'c$, etc. are drawn through the limits of the portions of the slices *1*, *2*, *3*, *4*, and *5* that come directly over the bearing rt of the concentrated load in (a). The portion of the trapezoid contained between each two consecutive lines represents the amount of load to be added to the respective loads *1*, *2*, *3*, *4*, and *5*. For instance, the weight represented by $a''bb'a'$ is the amount to be added to load *1*, $bcc'b'$, to load *2*, etc. The weights of these several loads a, b, c, etc. may be found by the following formula:

$$P = \frac{(k_1 + k_2)l}{2}$$

The results obtained are given in column 2 of the table on the drawing; thus, load a is equal to $\frac{(a''a' + bb')\,a'b'}{2}$.

In order to find the moment of these loads about the point a_1, Fig. 17 (a), it is first necessary to determine the distance of the centers of gravity of the respective loads from the edge r of the bearing. This is calculated by the formula

$$l_2 = \frac{(k_2 + 2k_1)l}{3(k_1 + k_2)}$$

The result found by using this formula is added to the distance rs, and the sum is the lever arm on which the load acts. For example,

$$l_1 = \frac{(a''a' + 2bb')a'b'}{3(a''a' + bb')} = \frac{(840 + 2 \times 880)1\frac{3}{4}}{3 \times (840 + 880)}$$

$= .88$ inch $= .073$ foot.

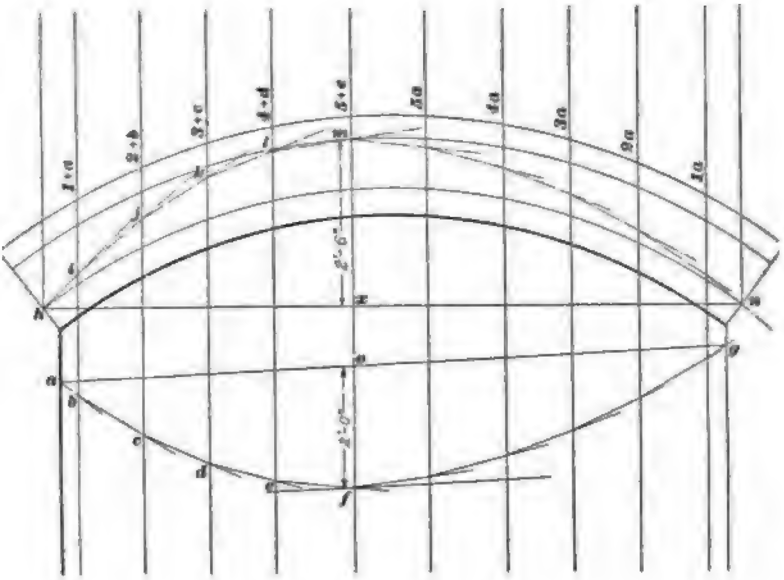
$rs + .073$ equals the arm of the load a. Multiplying the values in the second column by those in the third column, the total moment due to the load from the I beam is found to be 14,655 foot-pounds. To this must be added the moment due to the load of the masonry, or 28,517 foot-pounds, as found in Fig. 16, and the sum, 43,172, divided by $2\frac{1}{2}$ gives the crown thrust ab, Fig. 17 (c), of 17,269 pounds, or, in round numbers, 17,300 pounds.

On the vertical line ag, Fig. 17 (c), the loads $1 + a$, $2 + b$, $3 + c$, etc. are laid off. The line of pressure is then drawn according to the rules given. The final pressure bg on the joint at the skew back should be investigated in order to ascertain whether the arch is safe under the assumed conditions. From b, bh is drawn perpendicular to the joint of the skew back. This represents the perpendicular pressure on this joint applied at the point a_1, Fig. 17 (a). Since the angle hbg is less than 25°, there is no danger of the joint sliding. The depth of the arch ring being 18 inches, the distance from the intrados to the point a_1 is $4\frac{1}{2}$ inches, and allowing $\frac{1}{2}$ inch for the joint, disregarding tension, l_1 becomes $4\frac{1}{2} - \frac{1}{2} = 4$ inches. The pressure bh, or 23,340 pounds, falls without the middle third of the width of the joint, and the maximum pressure k_1 at the edge may be determined by formula **3**, *Statics of Masonry*, Part 1, which is $k_1 = \frac{2P}{3l_1}$. Substituting these values gives $\frac{2 \times 23{,}340}{3 \times 4} = 3{,}890$ pounds, which is the pressure per inch of width across the inner edge of the skew back. This divided by 17, the effective length of the joint, gives 229 pounds per square inch, which is in excess of the assumed compressive strength of 150 pounds per square inch; this indicates

that the arch should either be made thicker or set in cement throughout. This, however, is hardly necessary, for if the point of application of the crown thrust had been assumed as one-third the depth of the joint in from the crown and the inner limit of the line of pressure, at one-third the depth of the joint from the intrados, the pressure throughout the arch would have been increased, but the pressure per square inch at the skew-back joint would have been diminished.

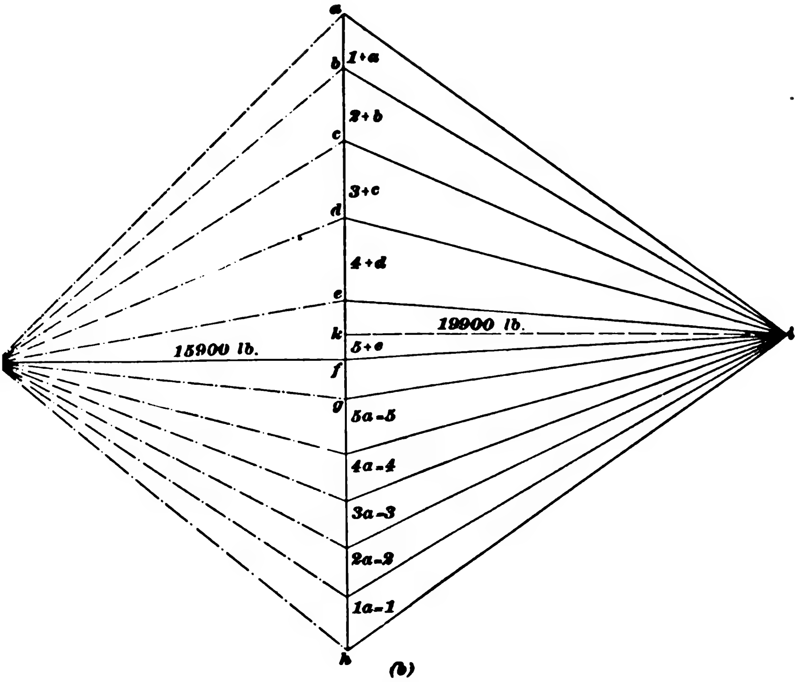
24. Unsymmetrical Concentrated Loads.—Probably the most difficult load to be taken care of in an arch is an unsymmetrical load. Such cases frequently occur, especially in railway construction, where the effect of a train passing over the arch must be considered. Examples of this character occur less frequently in building construction, but nevertheless it is necessary to know the effect of such loads. The important principles are shown in the following illustrative example:

The arch shown in Fig. 17 (*a*) is again considered, with the difference that there is but one I beam used instead of two, other conditions remaining the same. It makes no difference whether one or more unsymmetrical loads are taken, the same principles and methods apply. The full arch with its loads is laid out as shown in Fig. 18 (*a*). The next step is to find out where the line of pressure is horizontal; this is accomplished by the equilibrium polygon *a f g*. First the loads *a b*, *b c*, *c d*, etc., corresponding to the vertical loads *1* + *a*, *2* + *b*, *3* + *c*, etc., are laid out on the vertical line *a h* in (*b*). Some convenient point, as *i*, outside this line is assumed and connected to the points *a*, *b*, *c*, *d*, etc. At any point *a* on the reaction line in (*a*), *a b* is drawn parallel to *a i* and produced until it intersects the line of action of the load *1* + *a*. From this point *b*, *b c* is drawn parallel to *b i*, in (*b*), and produced until it intersects the load line *2* + *b*; this process is continued to the point *g*, where the last line intersects the line of the reaction; the points *g* and *a* are then joined. For convenience in plotting the curve, the loads *1* + *a*, *2* + *b*, etc. are assumed to act in line with the center of the slice.



1+a
2+b
3+c
4+d
5+e
5a
4a
3a
2a
1a
2'-6"
2'-0"

(a)



1+a
2+b
3+c
4+d
19900 lb.
15900 lb.
5+e
5a-5
4a-4
3a-3
2a-2
1a-1

(b)

Fig. 18

Next, a line is drawn parallel to *ag* in (*a*), and tangent to the equilibrium polygon. The point *f* at which this line is tangent is the point at which the line of pressure becomes horizontal; *fi* is now drawn in (*b*) parallel to *ag* in (*a*). The point *f* determines the amount of the reaction at either end of the arch. In order to find the intensity of this horizontal thrust, which, when scaled, equals 19,900 pounds, the perpendicular distance from *i* to *ah* in (*b*) is multiplied by 2 feet, the distance from *f* on the line *abcdef* to the point *o* on the line *ag* in (*a*). This product is divided by the vertical distance from the point *h* to *m*, the upper boundary of the middle half, which, measured on the vertical line through *f*, equals *xm*, or 2.5 feet. Then, *ik* in (*b*) multiplied by *fo* in (*a*) and divided by $xm = \frac{19{,}900 \times 2}{2.5} = 15{,}920$, or 15,900 pounds, in round numbers, the intensity of the required horizontal thrust. The distance *ik* in (*b*) should be measured to the scale with which the loads were laid out, and *fj*, equal to 15,900 pounds, is drawn perpendicular to *ah* at the point *f*, since the line of pressure becomes horizontal at this point. Then the force polygon is formed by drawing the lines *ja*, *jb*, *jc*, etc. From the point *h* in (*a*) *hi* is drawn parallel to *ja* in (*b*) and prolonged until it intersects the load line *1* + *a*, and *ij* is drawn parallel to *jb* until it intersects the load line *2* + *b*. This method is continued until the line of pressure is completed. The joint of rupture should be investigated as explained in Art. 22; in all shallow segmental arches, this is located at the spring line.

SEMICIRCULAR ARCHES

25. Semicircular arches are the same in principle as segmental arches, except that the joint of rupture varies. It is sometimes claimed that there cannot be an arch having more than 90° to 120° between the joints of rupture, but this depends on the amount of horizontal resistance offered by the spandrels. Below these limits, the arch ring is merely a skew back and could have horizontal joints, as in cases where vaults come together.

The following example serves to illustrate the necessary points:

EXAMPLE.—It is required to design a semicircular cut-stone arch having a span of 16 feet, a thickness of 2 feet, and surmounted by a rubble-masonry wall extending 6 feet above the crown.

SOLUTION.—By formula **2**, the depth of the arch ring should be $\frac{\sqrt{8+8}}{4} + .2 = 1.2$ ft., or 14 in.

The following method of calculation is commonly known as the *rational method*, being the one generally employed by engineers. The loads have been calculated for a thickness of 1 ft. The arch and masonry should be laid out as accurately as possible to some convenient scale and the masonry above the arch divided into sections S_1, S_2, S_3, as shown in Fig. 19. These strips should be narrower above the portion of the arch lying between radiating lines 15° and 45° with the horizontal, for this part contains the joint of rupture. The area of each strip in square feet, as given in column 2 of Table I, Fig. 19, is now calculated from the drawing. From the foot of each perpendicular ab, cd, etc., radiating lines are drawn representing the joints of the arch, and the area of each voussoir is then calculated from the figure. The cut-stone work is taken at 160 lb. per cu. ft. and the rubble work is 140 lb. per cu. ft. These values are given in column 3, and multiplying by the areas in column 2, the loads given in column 4 are obtained.

The center of gravity of each vertical load is now found. It is assumed that the centers of gravity of the loads S_1, S_2, S_3, etc. lie in a vertical line through the middle of the width of these areas. Although this method is not absolutely correct, especially in the cases of S_1, S_4, and S_5, it is, in this case, near enough for all practical purposes. The center of gravity of each voussoir is found by the method given in Art. **18**. The center of gravity of each load and of each voussoir being found, the center of gravity of the combined weight of the load and voussoir must be calculated. This is done by the use of moments; for instance, by finding the centers of gravity of the loads S_1 and V_1, and dividing the sum of the moments of these loads about the center line of the arch by the sum of the loads, or $(985.6 \times .58 + 202.4 \times .54) \div (985.6 + 202.4) = .573$ ft. This result is given in the last column of Table I on the drawing, and is the horizontal distance from the center line ab to the center of gravity of the combined loads S_1 and V_1.

The vertical loads being found, the horizontal loads must be calculated according to formula **3**, in which $h = \frac{1}{3} w d l$, the intensity of these loads being given in column 4 of Table II, Fig. 19. The values of d and l are taken directly from the drawing by scaling, and the

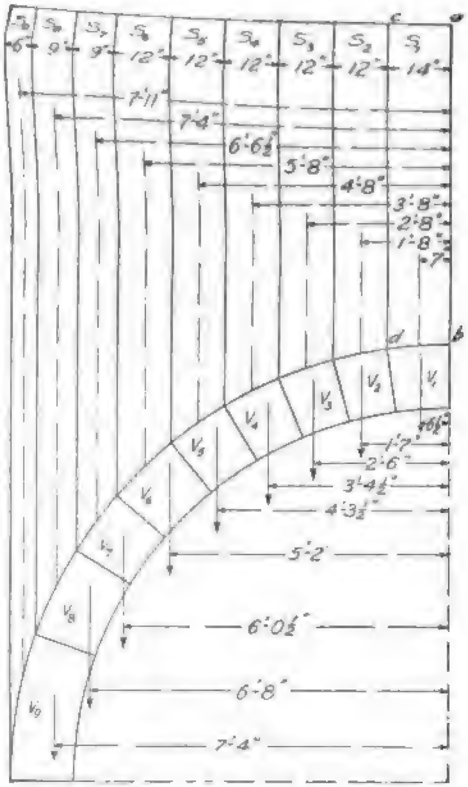


TABLE I

1 Load	2 Area in Square Feet	3 Weight	4 Load in Pounds	5 Arm in Feet	6 Moment	7 $\Sigma M_s \div \Sigma L_s$
S_1	7.04	140	985.6	.58	572	
V_1	1.265	160	202.4	.54	109	.573
S_2	6.1067	140	863.3	1.67	1,442	
V_2	1.07	160	171.2	1.58	270	1.65
S_3	6.4583	140	904.2	2.67	2414	
V_3	1.141	160	182.6	2.50	457	2.64
S_4	6.875	140	962.5	3.67	3,532	
V_4	1.217	160	194.7	3.38	658	3.62
S_5	7.4583	140	1,044.2	4.67	4,876	
V_5	1.325	160	212.0	4.29	907	4.60
S_6	8.2813	140	1,159.4	5.67	6,574	
V_6	1.52	160	243.2	5.17	1,257	5.58
S_7	6.96	140	974.4	6.54	6,373	
V_7	1.36	160	217.6	6.04	1,314	6.45
S_8	7.84	140	1,097.6	7.33	8,045	
V_8	1.849	160	295.8	6.67	1,973	7.18
S_9	6.083	140	851.6	7.92	6,745	
V_9	3.16	160	505.6	7.33	3,706	7.70

TABLE II

1 Number of Joint	2 Intensity of Vertical Force in Pounds L	3 Horizontal Distance of Point of Application of L From Crown	4 Intensity of Horizontal Force in Pounds H	5 Horizontal Thrust of Vertical and Horizontal Forces $\frac{\Sigma M_s}{\Sigma L_s} = H'$	6 Horizontal Thrust of Vertical Forces Only
1	1,188	.573	21	919	890
2	1,035	1.65	60	2,410	2,319
3	1,087	2.64	103	3,774	3,596
4	1,157	3.62	161	4,644	4,367
5	1,256	4.60	246	5,284	5,409
6	1,403	5.58	380	5,911	5,312
7	1,192	6.45	435	5,417	4,625
8	1,393	7.18	756	5,908	4,819
9	1,357	7.70	1,680	5,759	3,866

FIG. 19

result tabulated. In order to determine the maximum crown thrust, calculations must be made for each joint. This is done by taking the moments of the several loads acting on the voussoirs about the inner limit of the line of pressure and dividing their sum by the sum of the forces.

In Fig. 20, the horizontal thrust H' due to load *1*, acting about the point p_1, is equal to $S_1 + V_1$ times the perpendicular distance from its

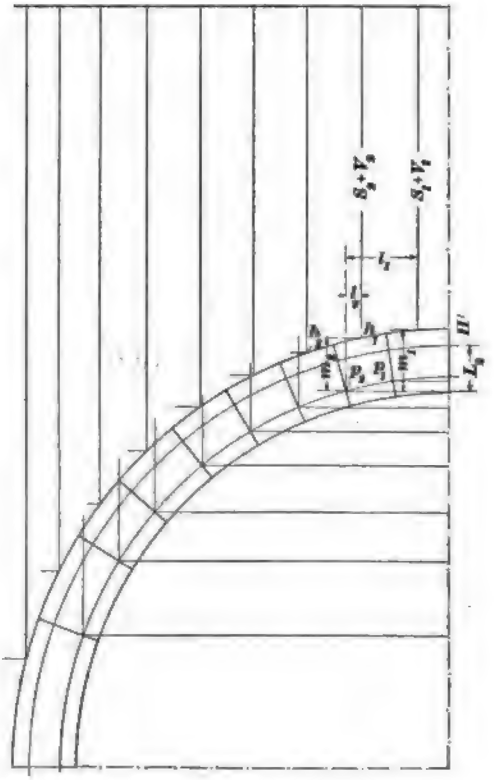


FIG. 20

line of action to p_1, plus h_1 multiplied by the perpendicular distance from its line of action to p_1, and divided by the perpendicular distance from the upper limit of the line of pressure H' to a horizontal line drawn through p_1. The horizontal thrust about joint *2* may be expressed by the formula

$$H_2 = \frac{(S_1 + V_1)\,l_1 + (S_2 + V_2)\,l_2 + h_1 m_1 + h_2 m_2}{L_2}$$

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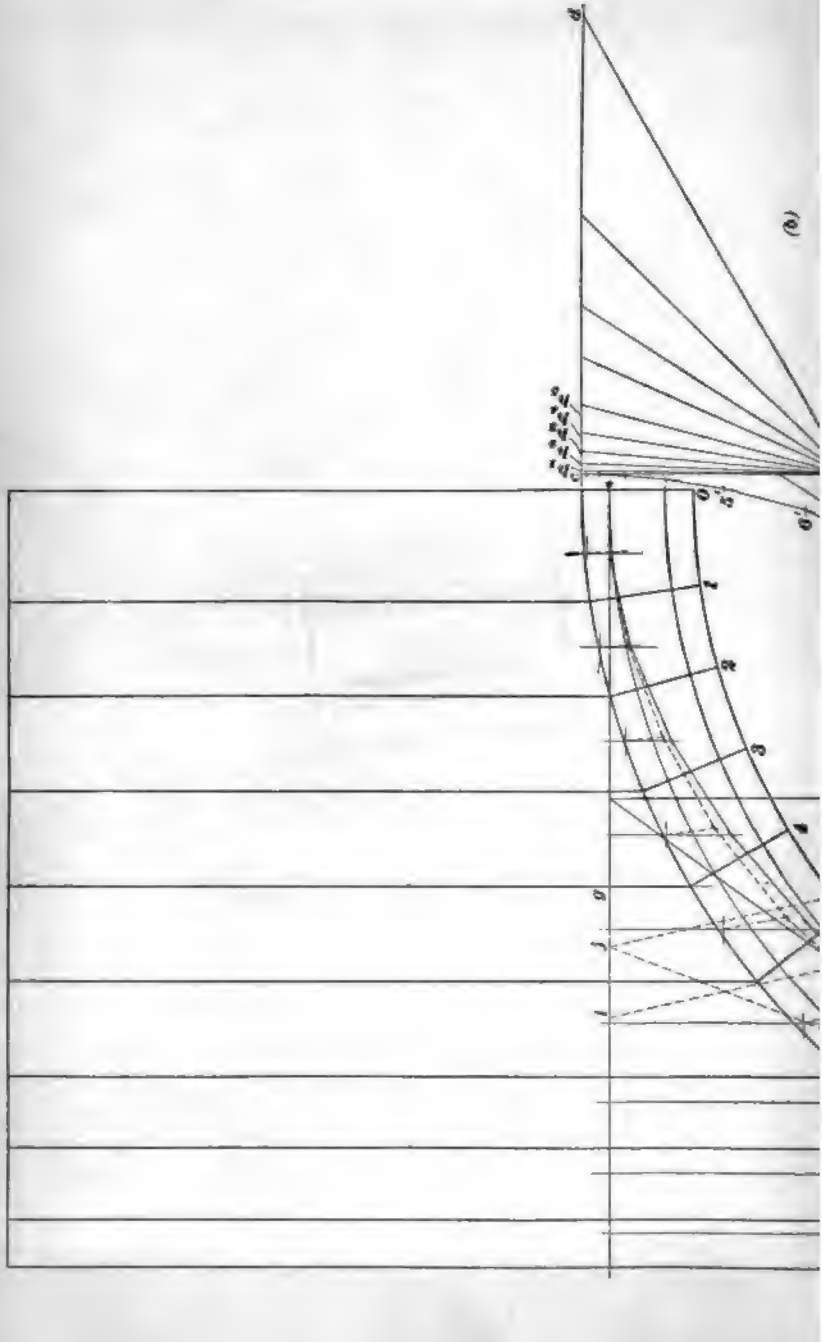

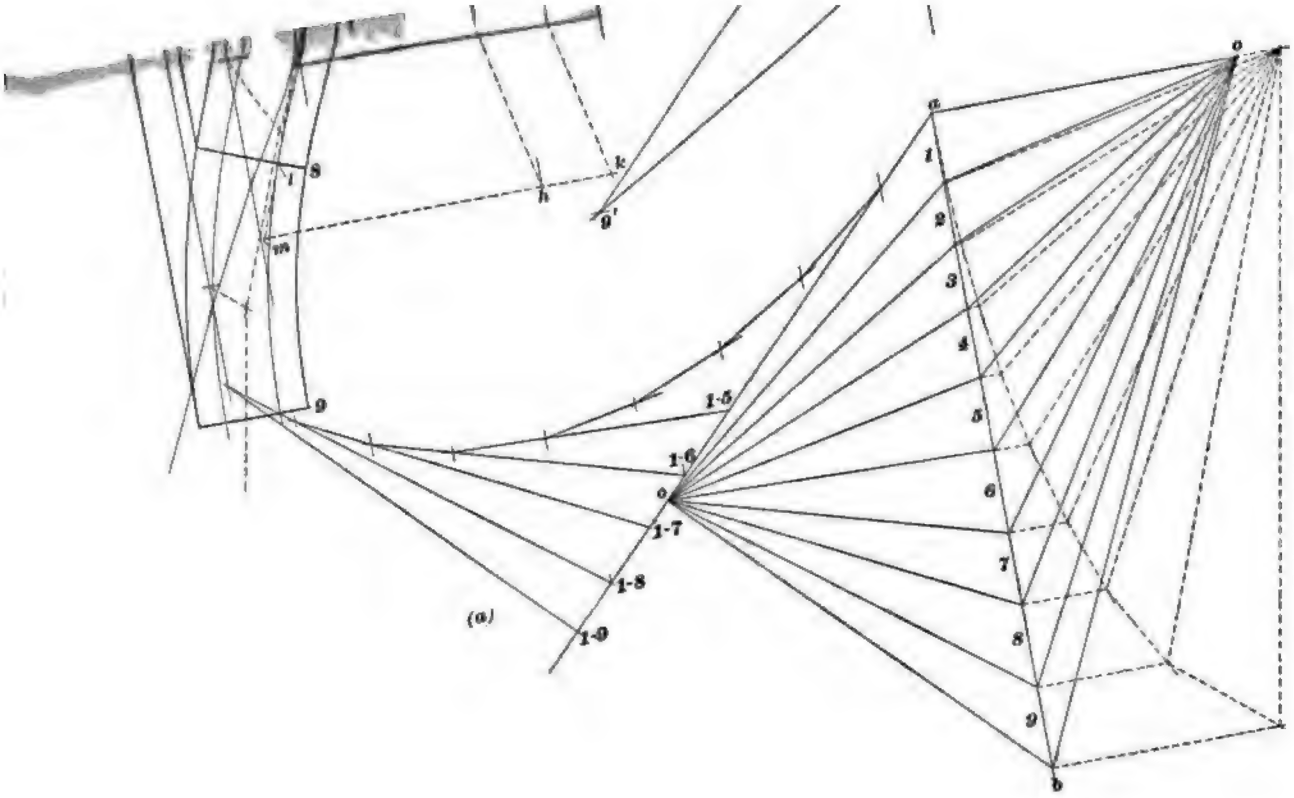

Fig. 21

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The result of a similar calculation for each joint is given in column 5 of Table II, Fig. 19. The values given in column 6 are for the same horizontal thrust of the arch, in which the forces h_1, h_2, h_3, etc. are neglected; that is, $\frac{(S_1 + V_1)\, l_1 + (S_2 + V_2) l_2}{L_2} = H_2$. Considering both the horizontal and the vertical forces, it can be seen from column 5 that No. 6 is the joint of rupture, while when only vertical forces are taken into consideration, it is No. 5, as shown in column 6.

26. The preceding calculations may be simplified by using the method given in Art. **22.** This is the same in regard to the vertical forces when considered alone, but a slight difference is introduced in considering both the horizontal and the vertical forces.

Referring to Fig. 21, the vertical loads *1, 2, 3, 4, 5*, etc. are laid off on the line *a b* in (*c*). From any point *o* outside of this line, lines *o 1, o 2, o 3*, etc. are drawn, and the centers of gravity of the combined vertical loads are found by the method shown in Fig. 12. In order to find the center of gravity of the horizontal loads, the several loads h_1, h_2, h_3, etc. are laid off on the line *c d* in (*b*); then the procedure is similar to that in the previous case. Finally the broken line *5′ 6′ 7′ 8′ 9′* is obtained. By extending the parts of this line to the line *e c*, the centers of gravity of the combined horizontal loads are found. In order to find the centers of gravity of the combined vertical and horizontal forces, it is only necessary to draw a horizontal line through the point found in (*b*) and a vertical line through the corresponding point in (*a*); thus, for joint *8*, draw a horizontal line through *1–8* in (*b*), and a vertical line through *1–8* in (*a*), thus locating the point *f* in (*d*). On a vertical line through *f*, a length *g h* is laid off equal to the sum of the vertical loads from *1* to *8* inclusive, and from *g* to *i* a horizontal force is laid off equal to the sum of the horizontal forces from *1* to *8*. Then the points *i* and *h* are connected and *j k* is drawn through the point *f* and parallel to *i h*. From the point *j*, the line *j m* is drawn through the inner limit *l* of the joint *8*. The length of the horizontal line *m k* will then be the horizontal thrust due to the vertical and horizontal forces acting on the joints from *1* to *8*. By this method, the horizontal thrust of each joint

throughout the arch may be found. The full lines throughout the figure refer to the calculations for the horizontal thrust considering the vertical forces only, and the dotted lines refer to the calculations for the horizontal thrust due to the horizontal and vertical forces.

It is found that the line of pressure for the vertical forces considered alone lies outside the middle half at joint 9, but the line of pressure due to the vertical and horizontal forces lies well within the middle half throughout the arch. It is probable that no arch would be in danger of failure if backed by an amount of masonry sufficient to resist any movement. This is especially true of arches built in solid masonry walls, but where arches are loaded with a concentrated load, great care should be taken that abutments of sufficient size are provided.

FLAT ARCHES

27. Flat arches are really segmental arches with the spandrels and tympanum filled in solid by extending the voussoirs to the spring line and bringing the spandrel masonry to a level with the crown. In Fig. 22 is shown a flat arch; the shaded portion represents the true arch that supports the load. Arches of this kind should always be built with a camber or slight rise, for two reasons: First, a practical one, since a camber allows for deflection of the arch when it is required to support a load; second, an esthetic one, for it prevents any appearance of sagging in the soffit.

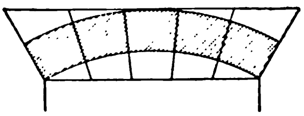

Fig. 22

Flat arches are not adapted to long spans, for as excessive depth is prohibited, it can readily be seen that the greater the span of the arch the less curve it contains. For instance, Fig. 23 shows a flat arch having a span of 10 feet and a depth of 12 inches, the rise of the arch being very slight, which produces an intense horizontal thrust at the abutments. These principles apply to all arches.

An elliptical arch may be considered as segmental with a special form of skew back; and those of horseshoe form, as

segmental with the skew back carried forward. A pointed arch is, in reality, two segmental arches leaning against each other, whose forces, being vertical, are calculated by the same principles. In the latter arch, the horizontal thrust is much less than in a circular arch, and the joints should be normal to the intrados. In most pointed arches, two centers are used, although in some cases as many as four or five may be employed. It is well to use radiating joints, since they are practically perpendicular to the line of pressure, and if, as in lancet arches, this rule is not adhered to, the voussoirs are liable to slide on one another, which is an indication of poor construction.

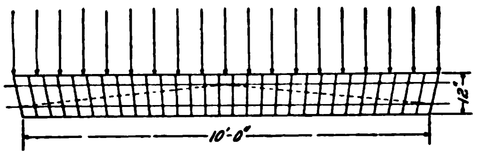

Fig. 23

ARCHES AND CONCRETE

28. The reenforced concrete arched bridge shown in Fig. 24 is built on the skew; that is, the axis of the bridge is not at right angles to the faces of the arch. The bridge was built to carry a roadway over a stream, and though the arch is not segmental, it is made up of portions of circles, a large radius being used for the crown of the arch and smaller radii at the spring line. The rise of the arch is 13 feet 9 inches, and its thickness at the crown is 2 feet. The essential elements of the arch consist of the abutments, the parapet walls, and the arch ring proper. The abutments are heavy masses of concrete mixed in the proportion of one part of cement to three and one-half parts of sand with seven parts of broken stone. The concrete in the parapet wall is composed of one part of sand, two parts of cement, and five parts of broken stone, while the arch ring consists of two parts of sand, one part of cement, and four parts of broken stone. The arch ring is reenforced with 1¼-inch Thacher bars, one of which is shown in view (c). These bars are about 30 feet in length and are joined with turnbuckles.

Throughout the arch ring, there are 25 tons of steel bars held in position vertically by means of strap iron, and placed as designated in views (*c*) and (*d*).

In constructing bridges of this character, it is considered advisable to lay all of the concrete at once, but if this is impossible, it is well to construct the bridge in parallel sections running lengthwise of the arch. By following this method of construction, the arch is not materially weakened, for each ring may be considered as a complete unit of the arch. The faces of the parapet walls and all finished walls are molded against finished tongued-and-grooved planking, while the back of the parapet wall and the rubble wall are molded against rough planking or timber. The sides of the mold are usually held against bulging by galvanized wire about the size of telegraph wire, inserted through the boards with loops, and twisted in order to make them taut. These wires are afterwards cut off, after which the mold is removed. In order to prevent the boards of the mold from warping when the wet concrete is deposited, they are painted with crude petroleum. This fills the pores of the wood and prevents the absorption of water from the concrete. When the work has been entirely constructed and the boards of the mold removed, the finished portion is worked over with carborundum blocks with a mixture of cement and water, the carborundum cutting down the high places and the cement mixture filling in the interstices.

In the design of the bridge shown, it was deemed advisable to make joints *a*, *a* in the parapet walls to allow for expansion. By dividing the work into such blocks, any irregular fracture that would otherwise occur is avoided and the work will crack along the line of least resistance, which is along the line of the joint.

The several details and the general design of the construction are shown on the drawing, which is fully dimensioned and needs no further explanation.

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FLYING BUTTRESSES

29. The **flying buttress** is a form of arch that is usually found in the cathedrals abroad and was originated for the purpose of taking up the outward thrust of the groined ceilings. It is usually started from a point opposite the joint of rupture of the groined ceiling and extends to a solid masonry pier forming a portion of the outer wall of the side aisle of the cathedral, as shown at *a*, Fig. 25. It was customary in structures of this nature to build a portion of the pier above what might be termed the *pitch line* of the flying buttresses, thus forming a sort of finial, which serves the apparent purpose of decoration, but in reality was added as an extra weight to counteract the outward thrust of the buttress.

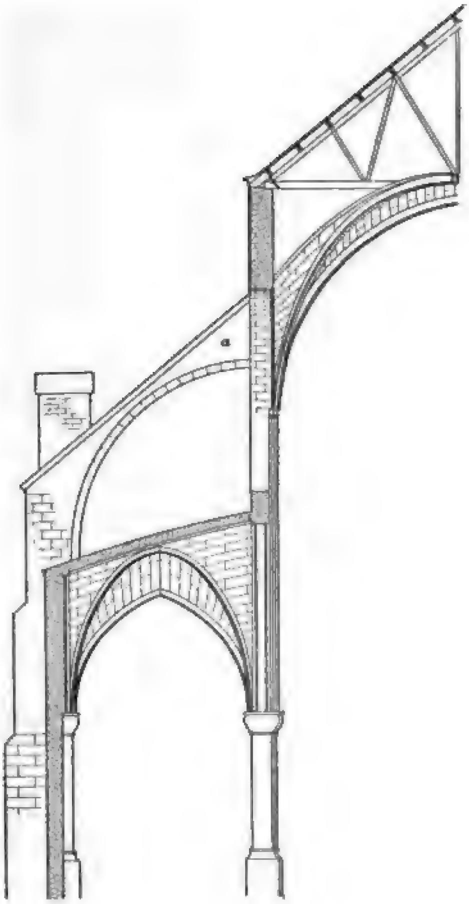
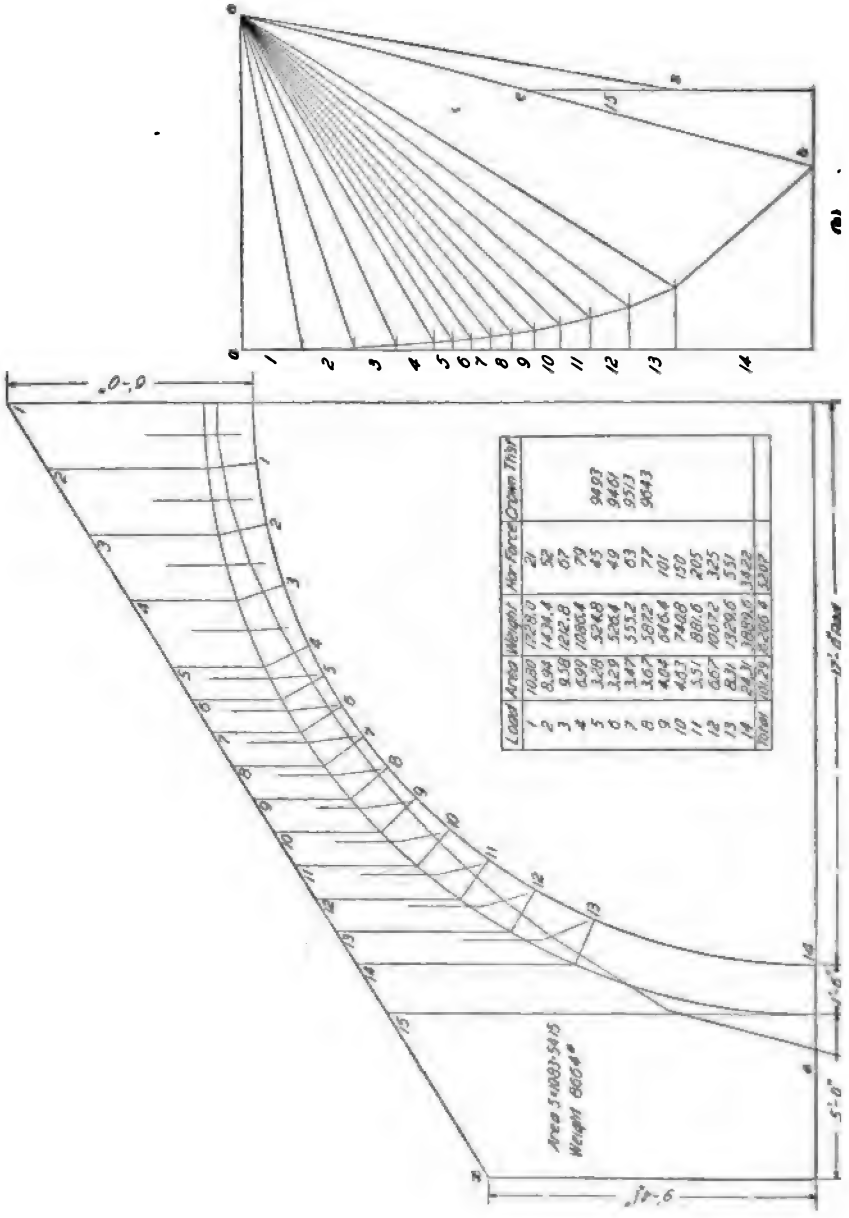


Fig. 25

30. In Fig. 26 is shown a flying buttress from a church in France. The radius of this buttress is 17 feet; the thickness of the arch ring, 18 inches; and the distance from the intrados to the top of the masonry above the crown, 6 feet.



Load	Area	Weight	Hor Force	Crown Thk
1	10.80	1728.0	21	
2	8.94	1434.4	52	
3	9.58	1212.8	67	
4	6.99	1086.4	79	
5	3.28	524.8	45	9493
6	3.29	526.4	49	9461
7	3.47	555.2	63	9513
8	3.67	587.2	77	9643
9	4.04	646.4	101	
10	4.63	740.8	150	
11	5.51	881.6	205	
12	6.67	1067.2	325	
13	8.31	1329.6	551	
14	24.31	3889.6	3422	
Total	101.29	16,206.4	5207	

The outer edge of the wall is located 6 feet 6 inches from the inner limit of the arch, and the lower end of the pitch line is located at x, which is 9 feet 4½ inches above the spring line. This buttress has no finial, but the base is made comparatively thick. If the formula for the thickness of the arch ring were applied, it would be found that there is a deficiency in the thickness of the keystone. In solving the problem, it is assumed that there is a horizontal force equal to $\frac{1}{2} w d l$, and although this is well enough in an arch put together with good mortar, it is doubtful whether such a force exists in cases of this kind. The wall above the arch is divided into fourteen loads, beginning at the crown. The first four loads have been made 2 feet in width; the following nine, each 1 foot in width; and the last, No. *14*, 1 foot 6 inches.

Column 2 of the table on the drawing gives the area of each of the loads, while in column 3 are the products of the quantities of column 2 and 160 pounds, the unit weight of the stone. Column 4 gives the amount of the horizontal force, as figured by the formula. The point of application of this force is at the middle point of the vertical projection of the back face of the voussoir. It is found by calculating the several moments that the greatest crown thrust is at joint *8* and equals 9,643 pounds; this is assumed to be the crown thrust of the arch.

Load *1* is now laid off vertically from a in Fig. 26 (b), and the horizontal force due to the first load laid off horizontally at the end of load *1*. Then load *2* is laid off, and from the second point the sum of the horizontal forces 21 and 52 is laid off horizontally. Similarly, load *3* is laid off, and from the third point the sum of the first three horizontal forces is laid off. By this method the load line $a\,b$ is found, which gives the resultant of the loads acting on each voussoir. The points in the line $a\,b$ are now connected to the point o, and from this polygon the line of pressure is drawn in (a). It is found that, even with the assumption of this horizontal force, the line of pressure is at the outer limit at joint *13* and is considerably outside the arch below this joint. However, the masonry being so well bonded at this point,

it acts with the arch, and the danger of overturning is averted by the weight of the masonry load *15*. In order to bring the load *15* into the diagram and find the center of pressure on joint *14*, the scale of the last line shown in view (*b*) has been reduced one-half, giving a point *r*, from which is drawn vertically the line *15* on which the load *15* is laid out to one-half the scale used in the rest of the diagram. The point *o* is connected with the point *s*, and if from the intersection of the center of gravity of load *15* with the final force of the arch in (*a*) a line is drawn parallel to *o–s* in (*b*), it will be found to intersect the joint *14* at the point *e*, which is sufficiently within the middle third of the base to be safe.

31. If the assumption of the horizontal forces had been neglected in this figure, the joint of rupture would have been found somewhere near joint *5* or *6*, and the line of pressure would have passed through the extrados at a much higher point. It is probable that considerable tension exists in this arch, although it stands and is in good condition.

This arch could be altered, in several ways, to draw the final force in the line of pressure within the arch; first, by raising the crown and making the arch more pointed, which would increase the vertical arm about which the horizontal thrust acts, thereby decreasing its intensity; and second, by increasing the thickness of the arch ring throughout, producing the same effect. A third remedy could be applied by increasing the loads from No. *10* to No. *14*. A fourth method of alteration would be to decrease loads *1*, *2*, and *3*. This is often done by introducing an opening, usually circular in form, into this part of the buttress. In this way the joint of rupture may be drawn down the arch to a lower joint and the line of pressure made to follow the contour of the arch ring more closely.

VAULTS

32. A **vault** is an arch projected along an axis whose shape, whether straight or curved, determines the class to which the vault belongs. Arches and vaults are used in different locations, arches usually being short and backed by masonry which they support, while vaults generally have little or no backing, support no masonry but their own weight, and almost invariably rest on walls or columns on which the stability of the vault is almost entirely dependent. They are, as a rule, employed only as ceilings, the roof proper being supported by a truss, although in some cases the vault forms the roof as well as the ceiling. There are two classes of vaults; viz., *solid* and *ribbed*. As shown later, the ribbed vault is simply an outgrowth of the solid one.

33. Solid Vaulting.—The simplest form of vault is that usually known as a **barrel vault,** shown in Fig. 27. This form may be considered as a series of arches in which the stones have been laid so as to bond with each other. Its line of pressure is the same as that for an arch of equal span and depth. It is also known as a *wagon*, *tunnel*, or *cradle vault*. When the axis is at an angle with the face, the vault is said to be *skewed*. When vaults are built on walls pierced by windows, between which they are braced by buttresses, the thrust of the vault above the windows should be carried to the buttresses by building an I beam at the spring of the vault. In figuring the thrust, one bay, or the distance from center to center of windows, should be considered, and the final thrust should not fall away from the center of gravity of the foundation more than the allowable limit. An *annular vault* is a barrel vault whose axis is either circular or elliptical.

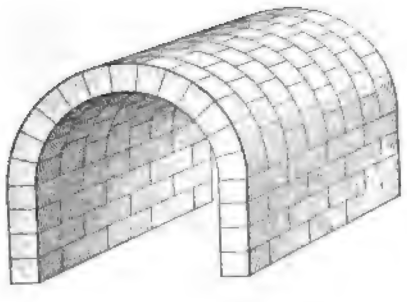


FIG. 27

34. A spiral vault, or one whose axis is a spiral, is usually employed to cover and support stairs leading up to towers, as shown in Fig. 28. Here the outer thrust of the vault is taken up by the circular outer wall and the inward thrust by the central column. To a great extent the steps form ties that hold the column in position. A great deal of ingenuity has been used in turning the vault. The outer half is made up of spiral courses, and the inner half of fan-shaped sections built on the inner column and sprung over to the crown of the spiral arch. This stairway ascends to one of the towers in the castle of Tristan l'Ermite, who was a provost in the time of Louis XI. The building probably dates back to the 15th century or earlier.

FIG. 28

35. Expanding vaults are those in which the end faces are of unequal size; these faces may be similar or of two distinct curves, as two circles, or a circle and an ellipse. Expanding vaults were largely used in Romanesque buildings to change a plan from square to octagonal. Such a vault is shown in Fig. 29.

36. Intersecting, or Groined Vaults.—When two barrel vaults of the same diameter come together their intersection is formed as in Fig. 30, the lines of intersection

being termed **groins.** The diagonal of this intersection *a b c* is a true ellipse. The problem of forming intersecting vaults was rather a complex one to the architects of earlier times, and many plans to overcome the difficulties encountered were adopted, one of which was to place a hemispherical dome over the intersection. An analysis of this intersection shows that if the joints were made in a line, as shown in the plan *a b c d*, the portion of the arch covering the intersection would fall. This proves that the triangles *a o b*, *o b c*, etc. are supported by the bond of the masonry and can be considered as corbeling out over the span. The intersection later formed a pointed arch, as shown in Fig. 31, and very often the vault was reduced to a series of vaults resting on piers, as in Fig. 32. In the analysis, the loads may be figured from the trapezoids *a b d c*, *c d f e*, *e f h g*, and *g h j i*, and the line of pressure for these loads found. It must be remembered, however,

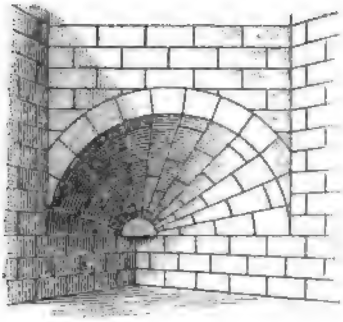
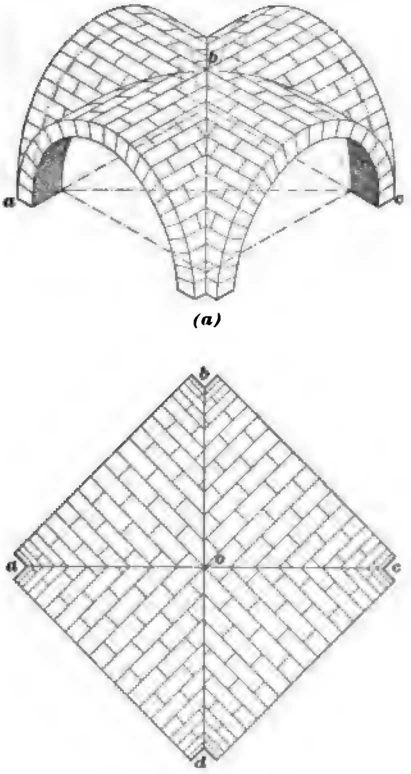


FIG. 29



(a)

(b)

FIG. 30

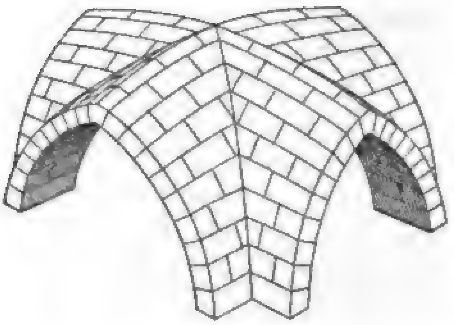


FIG. 31

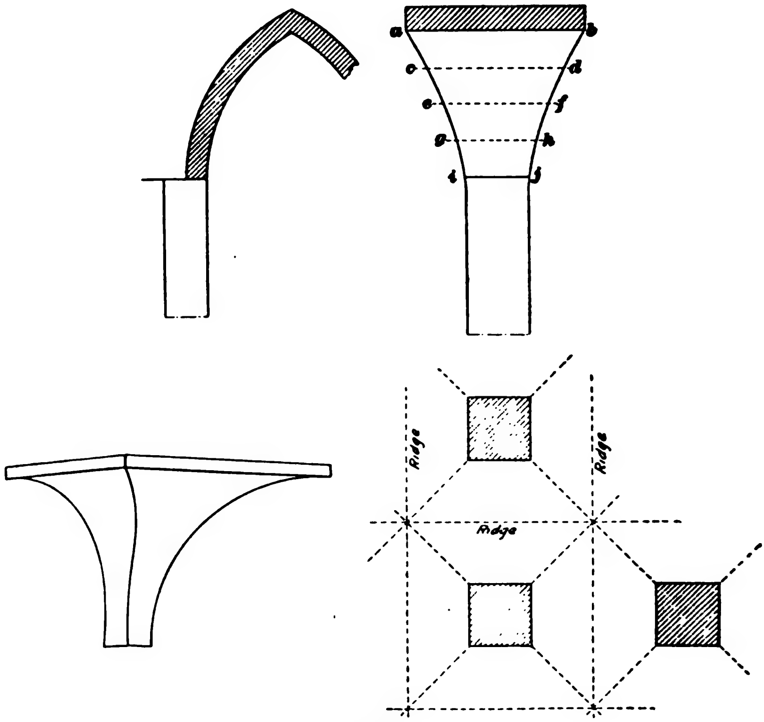


FIG. 32

that the haunches of this class of vault, when built as shown, cannot rise, for they are resisted by the other faces of the vault.

The introduction of ribs for supporting the vaulting was the forerunner of Gothic construction.

37. Gothic Vaulting.—In this construction the vaulting was made dependent on ribs for support by the following method of construction: Suppose that it is required to cover with a vault the square plan *a b d c*, Fig. 33, which may be two bays of a church. Connect *a d* and *c b* and describe the semicircle *c a b* on the diagonal. This represents the form of the diagonals *c g b* and *a g d*. The rise of the principal transverse ribs must be made less than the radius *c g* and more than half the side *a b*; this places the keystone of this arch at *k*, slightly below the keystone of the diagonals *a d* and *c b*.

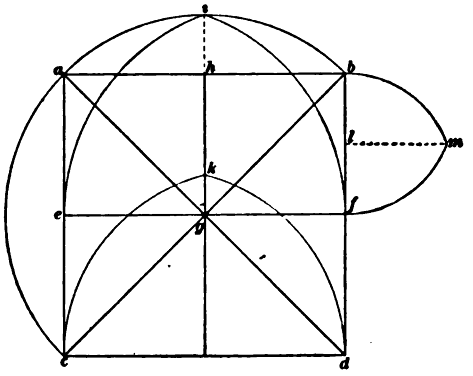


FIG. 33

The radius of the secondary transverse arch was made so that the keystone *i* would be at the same height as that of the diagonals. The wall arches were then made so that the rise was more than half *b f* and less than the height of *k*. Such wall arches were sometimes stilted, in order to raise the keystone *m*; Fig. 34 shows the ribs of such a vault with the covering removed. From this simple elementary form, the system was developed

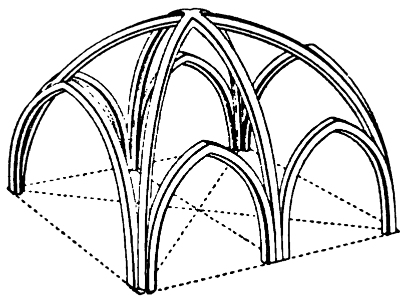
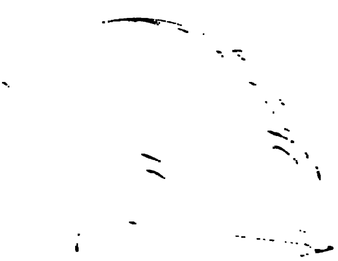


FIG. 34

[illegible] complicated vaultings, which it is not [illegible] discuss here.

[illegible] this same vault on which the covering has been placed. This vaulting should be made comparatively thin and all blocks should be made small enough so as to be easily handled.



JOINTS

38. The direction of the joints and shape of the stones in any vault is in [illegible] the stone cutter, who usually makes the [illegible] The subject of stereotomy will not, [illegible] entered into extensively. However, enough [illegible] enable the student to understand the [illegible]

[illegible] customary to make the joints radial, or [illegible] the curve of the intrados, and since the [illegible] joints in arches also apply to joints in [illegible] necessity for any further discussion con[illegible]

[illegible] Finding the Joints of a Skewed Arch.—Suppose [illegible] the parallelogram *a b c d*, Fig. 36, [illegible] curve of cross-section, perpendicular to [illegible] Draw the line *b f* perpendicular [illegible] equal to *a' g b*, and draw *a f* as [illegible] the distances from the line *f a'* to the [illegible] corresponding distances from *a' b* to [illegible] Similarly, draw *d e*, which completes [illegible] intrados of the vault. Since the [illegible] from the curved line, the former [illegible] for the transverse joints. Divide [illegible] the number of equal sections required; [illegible] for the sake of appearance.

Draw $e\,i$ through the corner e to a joint on the line fa, making it as nearly perpendicular as possible; this will give the direction of the longitudinal joints of the vault. The

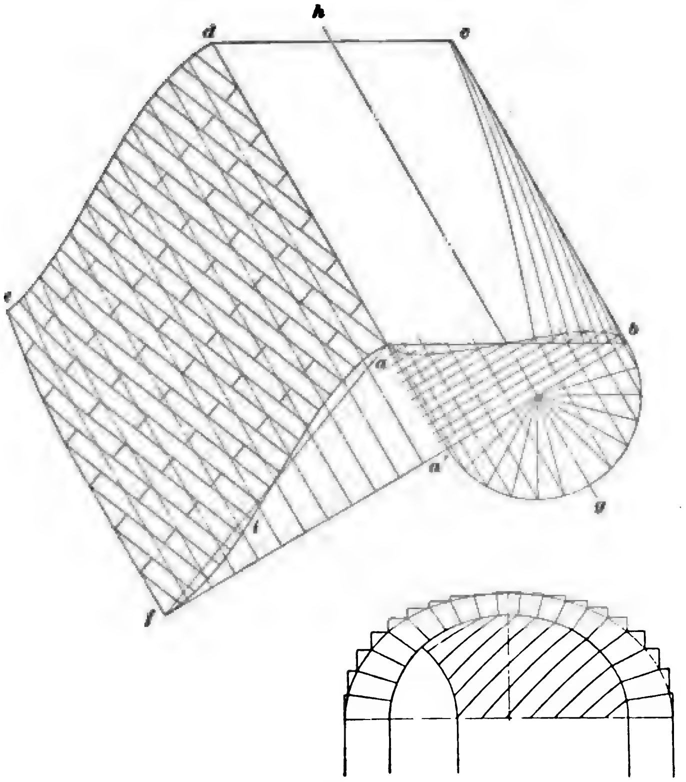


FIG. 36

number of joints from i to f will give the number of triangular pieces used at the foot of the vault along $a\,d$. The end faces of the arch are elliptical.

LOADS

40. The **loads** on simple vaults are the same as on arches; vaults, however, support no exterior loads and their stability depends chiefly on the piers and walls supporting them. The loads on ribbed vaults are usually left to the determination of the engineer. In fan vaulting the loads

increase toward the crown and must be figured accordingly, while in other kinds of vaulting they often increase toward the base; or, in other words, they diminish toward the crown.

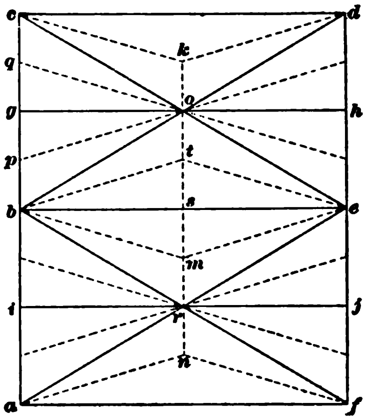


FIG. 37

In Fig. 37, let *afeb* and *bedc* be two bays of a Gothic church. The principal transverse ribs are shown at *af*, *be*, and *cd*. The rectangles are covered by the diagonal ribs *bf*, *ae*, *bd*, and *ce*, dividing the plan into triangles.

The triangles *arb* and *boc* are again divided by the secondary transverse ribs *ij* and *gh*. Next, *so* is divided into two equal parts, *st* and *to*, while *sr* is divided into two equal parts *rm* and *ms*. Then *t* and *m* are connected to *b* and *e*. The load on *be* is represented by the triangles *bem* and *bet*. By dividing *bg* into two equal parts *gp* and *bp* and connecting *p* and *o*, the two triangles *bop* and *gop* are formed, and similarly the triangles *goq* and *coq*. The load on the half diagonal rib is the weight of the two triangles *bop* and *bot*, and the load on the half secondary transverse rib is *pog* and *qog*.

From this plan view, a load diagram may be plotted as shown in Fig. 38, which represents the transverse rib *be*, Fig. 37. The section *dab* represents the sum of the triangles *mbs* and *tbs*, Fig. 37. A load diagram for the secondary rib would be the reverse of this, the heavier part resting on the lower portion of the arch. The thrusts of the triangles *mbe*

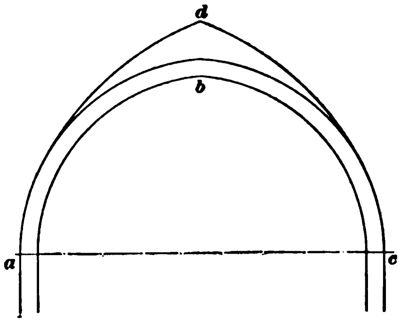


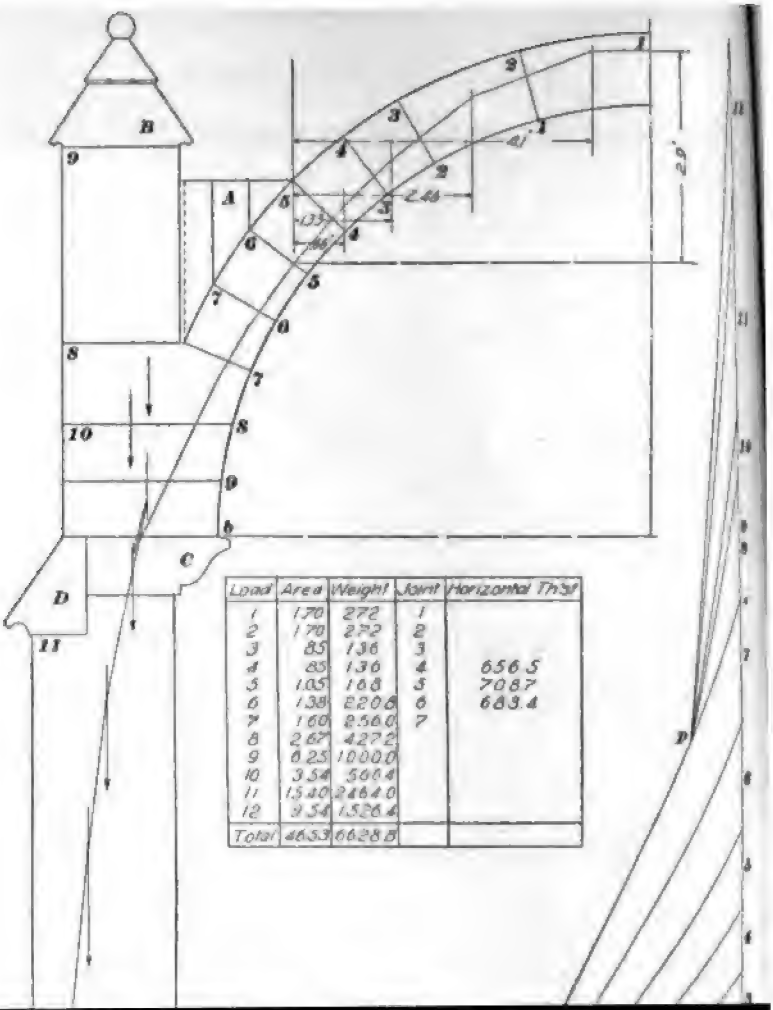
FIG. 38

and *t b e* in Fig. 37 counteract each other, producing a vertical load on the rib *b e*. The thrusts of the triangles *p o b* and *b o t* also counteract each other, producing a vertical load on the diagonal rib *b o*. The outward thrust of the rib *b s* with the load acts on the wall at *b*, tending to overthrow this wall. The diagonal ribs *b o* and *b r* act obliquely at the point *b*; they are, however, similar and their thrusts are equal. The components of these oblique thrusts in the direction of the length of the building counteract each other, but the components in a direction perpendicular to the length of the building are added. The resultant thrust at the point *b*, tending to overthrow this wall, is therefore equal to the sum of the thrust of *b s* and the transverse components of *b o* and *b r*.

41. In churches where this construction was used, flying buttresses were placed against the points *a*, *b*, *c*, *d*, *e*, and *f*, Fig. 37; an illustration of this case is shown in section in Fig. 25. When there were no side aisles, a plain buttress was used in place of the flying buttress, except in churches with high naves. The outward thrust of the secondary transverse rib is not very great, for the greatest load acts on the haunches, throwing the line of pressure very near the vertical.

Vaults are sometimes built of a series of ribs and panel fillings; these ribs should be figured for a complete section, taking the load on one panel from center to center of panels. A few examples will serve to illustrate the methods employed to find the stability of these vaults.

42. It is required to cover with a vault a space 13 feet 3 inches between walls whose height from the foundation to the spring line is 11 feet 6 inches. The vault is made a 12-foot span and supported on a corbel *C*, Fig. 39, in order to throw its thrust and weight on the inner edge of the wall, thus tending to maintain equilibrium. According to previous rules, the arch ring is made 1 foot in thickness. It is evident that the joint of rupture will be located somewhere near joint *5*. If the weight on the arch at the back of loads *5*, *6*, and *7* be increased by filling in the spandrel up to joint *4*, the line of



Load	Area	Weight	Joint	Horizontal Th'st
1	1.70	272	1	
2	1.70	272	2	
3	.85	136	3	
4	.85	136	4	656.5
5	1.05	168	5	708.7
6	1.38	220.8	6	683.4
7	1.60	256.0	7	
8	2.67	427.2		
9	6.25	1000.0		
10	3.54	566.4		
11	15.40	2464.0		
12	9.54	1526.4		
Total	46.53	6628.8		

[redacted]

pressure will be thrown down and consequently the horizontal thrust will be lessened. Beyond joint 7, the joints of the stones are made horizontal because the arch ring acts as a skew back up to this point and the stones *8* and *10* act also as a corbel. In order to oppose the force on joint 7, tending to overturn the upper part of the wall, the portion *B* is built on the stone *8*. It is assumed that the joint of rupture has been located, and the horizontal thrust has been calculated; also, that the intensity of the force *os* has been obtained, which is equal to a thrust of 1,622 pounds. It will be noticed that the loads *8*, *9*, *10*, *11*, and *12* have been laid off in the load diagram to a smaller scale than the other loads, because, if laid out to the same scale, they would necessitate a very large diagram.

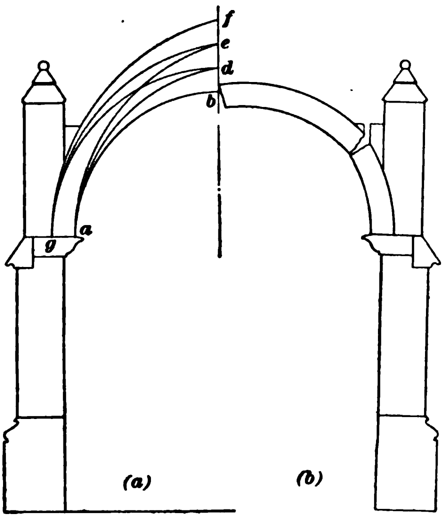
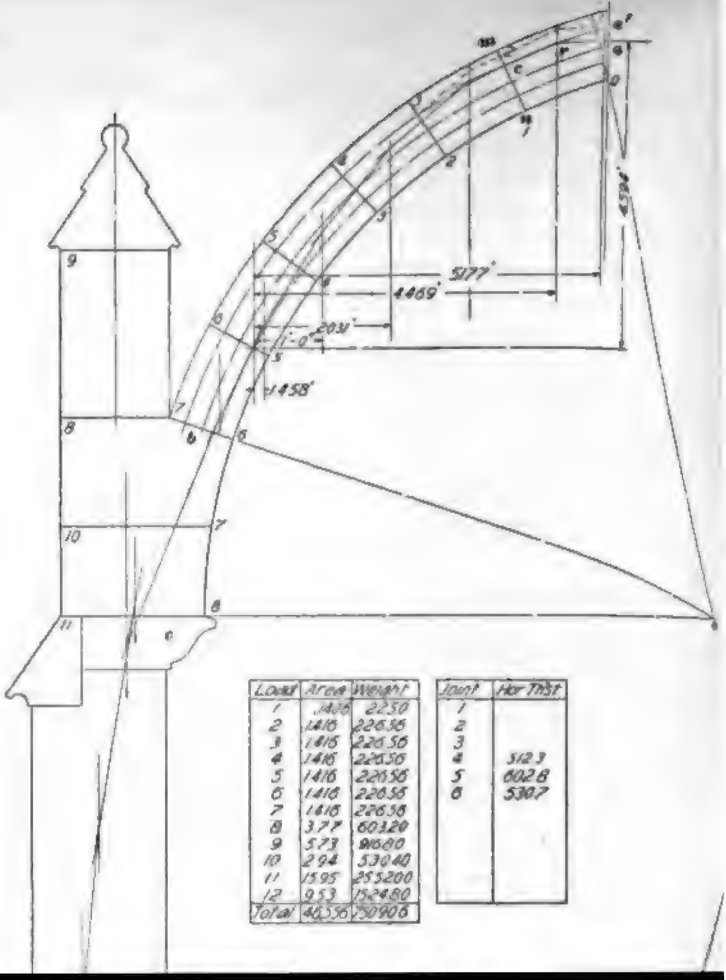


FIG. 40

The effect of the loads *8* and *9* is to throw the line of action toward the center of the wall. Following this thrust to the foundation, the final thrust is found to pass through the lower joint *ef* less than one-fourth the width from the edge *e*, which is not allowable under the assumption. This can be remedied by making the portion of the wall *B* heavier, or by increasing the spandrel load *A*, the corbel *C*, or the thickness of the wall. In other words, the design must be changed so that the final thrust falls within the middle half at least, or better, the middle third.

43. The early Romanesque builders found that the walls were pushed out and that the crown of the vault had a tendency to fall in, as shown in Fig. 4 (*c*), and more fully



Load	Area	Weight
1	.1406	2250
2	1416	22656
3	1416	22656
4	1416	22656
5	1416	22656
6	1416	22656
7	1416	22656
8	3.77	60320
9	5.73	91680
10	2.94	53040
11	15.95	255200
12	9.53	152480
Total	46.556	750906

Joint	Hor Thst
1	
2	
3	
4	512.3
5	602.8
6	530.7

in Fig. 40 (*b*). From these facts they learned that the arch should be built more pointed in order to reduce the outward thrust; hence, in the later period of Romanesque architecture, the arches were designed so that the extrados of the arch ring, instead of being semicircular, formed the diagonal of the new arch ring, which is shown at *a d e g*, Fig. 40 (*a*), which represents one-half of a semicircular vault. Sometimes the architects went still further and took the diagonal of this second arch as the intrados for the new arch, as shown at *a e f g*. These changes constitute one of the most important steps in arch construction.

44. The wall given in Fig. 39 is again shown in Fig. 41, supporting a Gothic or pointed arch instead of a semicircular arch. The radius of the intrados is 8 feet 3 inches. The depth of the arch ring is the same as in the previous case, being found by formula **2.** The line of pressure is found by the following method: The intrados is divided into a number of equal parts, in this case eight. The first two joints above the wall are horizontal, but since the thrust is now great enough to move the stones, the second stone *8* forms the skew back and also the bed for the built-up portion. The areas of the portions *1*, *2*, *3*, *4*, *5*, *6*, and *7* are calculated and given in the second column of the table on the drawing, the small triangular piece *1* being taken as a separate load. The horizontal thrust at the crown is assumed to be one-fourth the depth of the arch from the extrados. The voussoirs *2*, *3*, *4*, *5*, *6*, and *7* being similar, the centers of gravity lie at an equal distance from the center of the arch. The center of gravity of one of these voussoirs is found by the method already given. Then with a radius *o r* equal to the distance from the center *o* used in describing the curved intrados, to *r*, the center of gravity of the voussoir, the arc *a b* is described. If the length *c r*, which is equal to the distance from *c*, the point where the arc described by *o r* intersects *m n*, the joint of the voussoir, to *r*, the center of gravity of the voussoir, is laid off from each corresponding intersection of arc and joint, the center

of gravity of each voussoir may be obtained. The center of gravity of the combined voussoirs may be located and the horizontal thrust obtained by the methods previously given. The joint of rupture of the arch ring having no masonry backing lies at an angle of about 35° with the horizontal. In the last example, this angle was about 37° 40′, and therefore it is likely that the joint of rupture for the arch will be joint *4*, *5*, or *6*. Assuming the horizontal thrust to be one-fourth the depth of the arch ring from the top of the crown, the line of pressure may now be drawn. It is seen that the first part of this curve of pressure lies wholly without the limit, as shown by the dotted lines, and as it is necessary for it to fall entirely within the limit, a point *a* should be found below *a′*, through which a line lying wholly within the arch may be drawn; by trial, this point may be definitely located. The thrust corresponding to this point *a* is found, and a curve obtained that lies wholly within the inner half of the arch ring. The remainder of the calculations for this arch are similar to those in the previous example. It should be noted that the horizontal thrust of the pointed arch is con siderably less than that of the semicircular, for whicl reason the curve of pressure passes through the joint *ef* a the foot of the wall well within the middle half, showin that the design is stable. The total area of the pointed arc is also slightly less than that of the semicircular, and henc the former requires less masonry per foot of length. Thi

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thickness at the crown is slight, the vault would undoubtedly stand and there would be no reason to fear displacement were it not for the concentrated load of the pedestal placed over the crown of the vault.

As is often the case, where it is desired to reduce the weight on an arch, relieving arches or steel beams are built above it. This practice is common in relieving window lintels of the strain. In the present case a concrete floor has been used, which is reenforced by two 6-inch channels at either side, connected by $3'' \times \frac{7}{16}''$ bars placed 12 inches on centers. The floor acts as a beam in supporting the load. The spaces around these bars are filled with concrete, as are also the spandrels of the vault.

DOMES

47. The term **dome** in this Section is applied to the roof of a building, having curved surfaces and springing from a circle, ellipse, or any regular convex polygon. The term

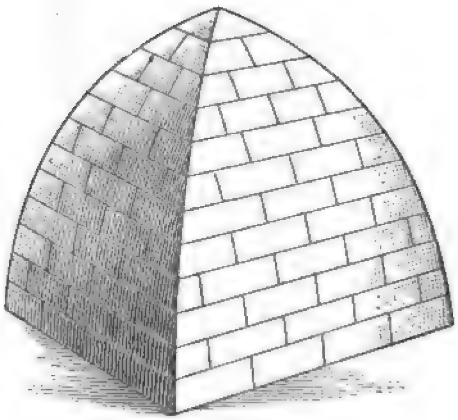

FIG. 44

may be further defined as including all roofs made up of bent triangles whose apexes meet in a point at the crown and whose bases are in the same plane. They may be truncated to admit a cupola or lantern, or they may be cut by the

planes of the walls, thus cutting off the bases of the triangles, but the underlying shape is the same.

Fig. 44 shows a dome built on a square plan and made up of four triangular pieces. This is practically the simplest form of dome, since the triangle is seldom, if ever, used as a

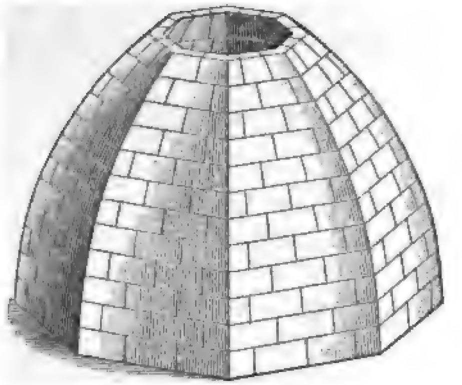

FIG. 45

plan. If the number of sides be increased by cutting off the corners of the square plan, the form shown in Fig. 45 is obtained. If the number of sides be increased infinitely, the true dome, such as was built on the Parthenon or St. Peter's Cathedral, Rome, is developed.

48. A dome cannot fail by the sides falling inwards because the individual voussoirs in each ring rest against each other, but it may readily fail by the spreading of the abutments; or, if thin throughout, but having strong abutments, there is danger of failure from spreading of the ring or from falling of the crown.

HEAVY FOUNDATIONS

SPREAD FOOTINGS

INTRODUCTION

1. The term **spread footings** is applied to the class of foundations illustrated in Figs. 1 and 2. These foundations are best adapted for the substructure of high buildings erected on soil of a clayey nature, and are especially necessary where the foundation stratum is of soft clay underlaid

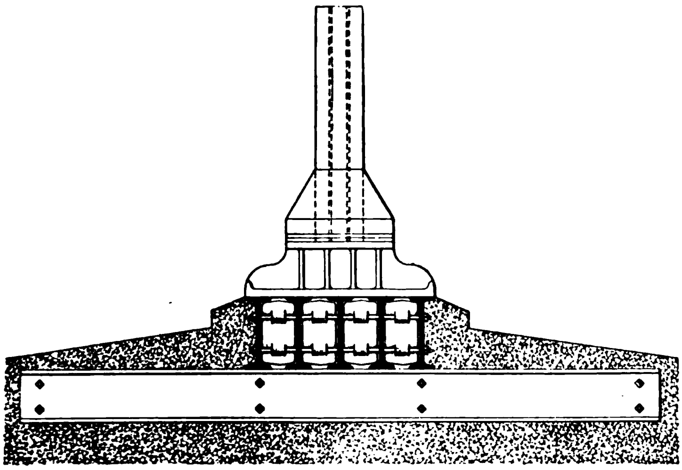

FIG. 1

with quicksand and the bed rock is so far below the foundation bottom as to preclude the possibility of driving piles to a bearing on it. These foundations may also be used to advantage where the stratum of clay is thin and its bearing

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capacity cannot be improved by piling. With such a soil there are many considerations that require careful attention and study and involve the incorporation of several features new to the usual stepped masonry foundations.

Tall buildings of the skeleton-construction type concentrate great loads on the basement columns; these, in turn, must be supported by footings of considerable area. As the exterior columns in this type of building support the outside, or curtain, walls, they are more heavily loaded than the interior columns; and since some settlement is sure to occur, it is desirable that this settlement be uniform. The simplest

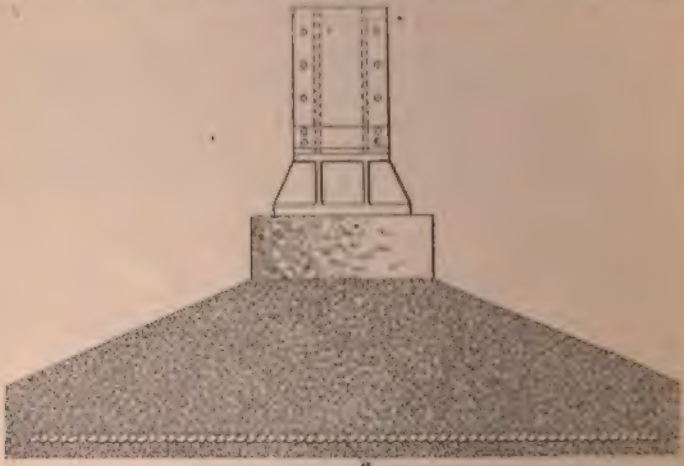
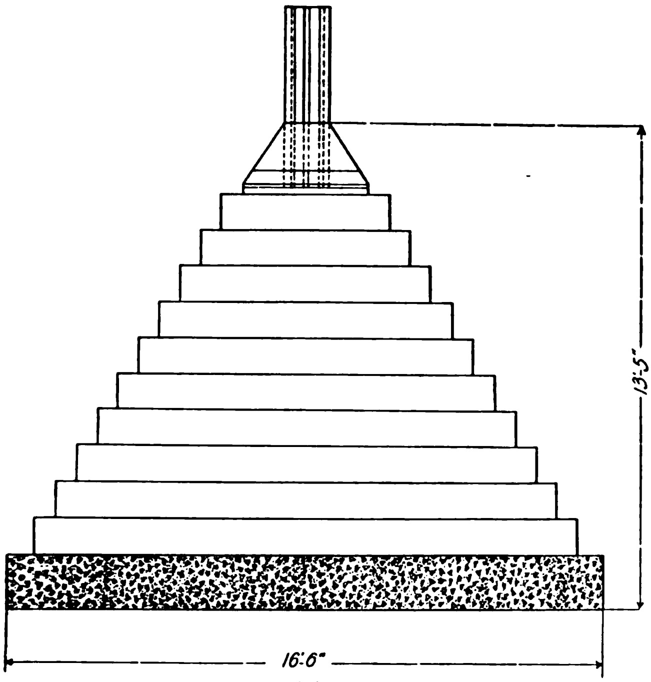
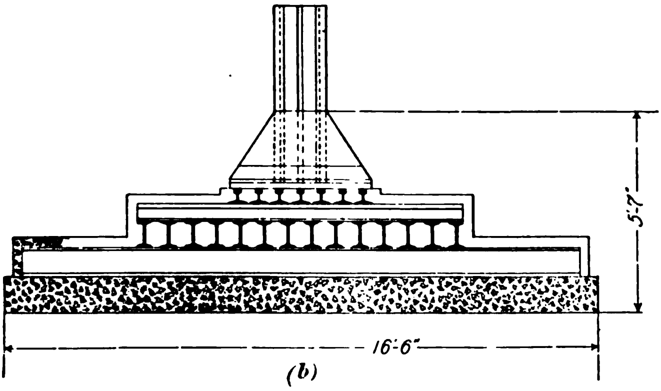


FIG. 2

way to attain uniformity in settlement is to provide a separate footing for each column or set of columns that act in unison, the entire building being in this way supported on isolated footings, that are accurately proportioned for the loads they must sustain. If these footings were of the usual masonry type, shown in Fig. 3 (*a*), having the proper proportions and batter, they would be of great depth and of such dimensions as to materially lessen the floor space in the basement of the building. This amount of masonry would add greatly to the unit weight on the soil and form a considerable percentage of the entire load from the column.



(a)



(b)

FIG. 3

2. A recapitulation, then, of the requirements that lead to the adoption of spread footings may be stated as follows:

1. That sufficient bearing may be obtained for the great loads requiring support in the modern tall building resting on a plastic or unstable soil.
2. That the foundation and footing may be so shallow as not to penetrate the stratum of clay and impair its bearing value when underlaid with quicksand.
3. That no foundation piers of great bulk occupy space in the basement that may be used for engine rooms or even bring in a good rental as cafés or shops.
4. That the weight of the foundation shall be so small a percentage of the entire load on the column it sustains, that a considerable portion of the footing area will not be taken up in carrying the weight of the foundation.
5. That the cost of the foundation shall not greatly exceed the cost of the usual stepped footings or foundation piers of masonry.

The second and third stipulations are of considerable importance, and the advantages that the spread footing possesses over the ordinary masonry footing, especially with regard to space saved in basement and the depth required for the respective constructions, are illustrated in Fig. 3 (*a*) and (*b*). In view (*a*) is shown the ordinary masonry pier with a concrete footing, while in (*b*) is illustrated a spread footing designed for the same load and conditions of soil. By comparing these views, it will be observed that nearly 8 feet in depth is saved by the use of the spread footing, provided that the basement floor is level with the top of the column base, and that the necessary pier in (*a*) occupies a considerable portion of the basement area if its foundation is only 5 feet below the basement floor.

3. Spread footings may be classified as ***steel-beam grillage*** and ***reenforced concrete foundations***. These two methods of construction are shown in Figs. 1 and 2, respectively. They both attain the same results; namely, obtaining a footing shallow in depth and having a large bearing area on the

soil as well as possessing sufficient transverse strength to transmit a great pressure from the comparatively small area of the column base to the large area of the foundation footing. In the steel-beam grillage foundation, the transverse strength is supplied by the resistance offered by the steel beams to bending, while the reenforced concrete footing is provided with the necessary transverse resistance by embedding iron or steel tension bars in the lower portion of the concrete, as designated at *a*, Fig. 2. It is necessary to provide metallic reenforcement to concrete structures subjected to transverse strength, because concrete has little tensile resistance compared with its compressive stress. Economical design, therefore, demands that this necessary tensile strength be imparted to the concrete, so that the neutral axis will be near the center of the section and the full compressive resistance of the material will be realized.

STEEL-BEAM GRILLAGE

4. The beams originally used in the construction of grillage footings were steel rails crossed in alternate layers. Undoubtedly they were adopted on account of being readily obtainable and also because of their shallow depth and the considerable resistance they offer to transverse stress. The top layer of steel beams, those directly under the cast-iron cap, usually have as much projection as any of the successive layers, and being fewer in number, are required to offer greater resistance to bending; so that, in most cases, steel I beams with a greater section modulus must be used in this position. Now that rolled structural shapes are obtained with facility, it is more economical to use I beams throughout the footing, where its height is not closely limited, as their weight is less for a given section modulus than steel rails. Another advantage exists in the fact that they may be readily secured together with anchor bolts and separators.

Where the soil is of a soft and plastic nature, three or more tiers of beams are used in grillage footings; but where

the loads are light and the soil is of a more stable character, two tiers of beams will usually be sufficient. Grillage foundations are either square or rectangular, depending on their location in the footing area. For the support of a single column the square foundations are preferable and more economical; the rectangular footings are used where the area available is narrow and the necessary area must be provided by lengthening the footing.

5. The usual design for a heavy grillage footing is shown in Fig. 4. It is necessary to provide first a bed of concrete *a* from 12 to 18 inches in thickness, tamped in two or three successive layers and composed of one part of Portland

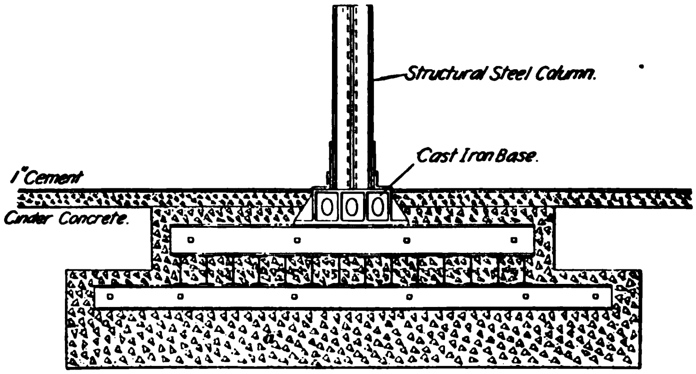


FIG. 4

cement, two parts of sand, and five parts of broken stone. On this concrete bed, after it has obtained its initial set, the first layer of I beams is placed, the spaces between the beams being solidly tamped with concrete. The beams in each tier are thoroughly secured to each other by means of separators and bolts. The separators may be of either cast iron or pressed steel; pipe separators should not be used. They should be placed not more than 6 inches from each end of the beam, and one should be placed under each of the outside beams in the tier above. Other separators should be introduced throughout the length of the tier so that the distance between separators will in no case exceed 5 feet.

On the first tier of beams the second layer of steel beams is crossed; and after these have been filled in with concrete, the third layer is placed in position and embedded in concrete. The concrete in all cases is placed on the tops, sides, and ends of the steel beams to a depth of from 4 to 6 inches, so that the entire steelwork is completely embedded. Before the steel beams are placed in position, they should be thoroughly painted with several coats of some good preservative paint, the steel being thus protected from corrosion until after the initial set of the concrete has taken place. When this precaution is adopted the steel will last indefinitely, provided the concrete contains an excess of cement. In placing the steel beams they should never be spaced closer together than 3 inches in the clear between the flanges, so that the concrete may be thoroughly rammed between them.

6. In designing steel-beam grillage foundations supporting columns, it is first necessary to ascertain the load on the column that is transmitted to the footing. It is also necessary to determine the area required for the footing in order that the allowable unit pressure on the soil may not be exceeded. When the area of the footing, and consequently its dimensions, has been determined, the lengths of the steel beams are known. The number required, their size and weight, must, however, be ascertained by calculation.

The steel beams that provide the necessary transverse strength for the footings are subject to failure by the crushing or buckling of the web or by bending. Since the webs of the beams are secured together at close intervals by separators and bolts, and because concrete is thoroughly tamped between the beams, there is little liability of the webs bulging or crippling, so that it may be considered safe from failure if the unit stress on the web does not exceed 10,000 pounds. Some engineers advise considering the web of the beam as a column and thus determining the allowable unit compressive stress. When column formulas are applied in this manner to determine the resistance of the web to bulging, the height of the web is considered as the

length of the column and should be taken as the distance between the fillets.

The proper method for determining the bending moment to which the steel beams in the grillage foundation are subjected has been a matter of some dispute among engineers. The practice among some is to consider the projecting portion of the beam, that is, the length beyond the edge of the cap or the successive layer above, as a cantilever, while others recommend as better practice that the entire length of the beam be considered and the bending moments calculated accordingly. It is doubtful whether either method gives the true value of the transverse strength of the successive tiers of the footing, for the beams are necessarily reenforced by the concrete. By disregarding the concrete, however, the problem becomes greatly simplified and any error is on the side of safety. The formulas for determining the bending moment on the steel beams in a grillage foundation may be derived as explained in the following pages.

FOOTINGS SUPPORTING ONE COLUMN

7. In Fig. 5 is shown the plan and elevation of a grillage footing. The area has been determined and the dimensions of the sides obtained. The pressure transmitted to the footing through the column is likewise known, and may be

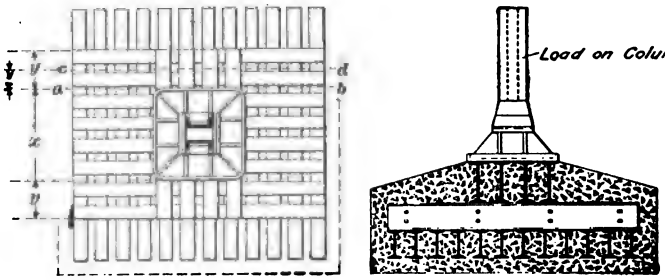


Fig. 5

designated as W. The dimensions of the cast-iron base plate are practically fixed by the design of the column; at any rate, they may be originally assumed, and reduced or

increased as conditions warrant. It is therefore considered that the distances y, y, and x in the plan are found, so that the length for the first tier of beams is equal to $x + y + y = x + 2y$. The load on these beams, since they transmit the entire weight on the column, is equal to W, and the load on each lineal foot of the tier is, in consequence, equal to $\frac{W}{x + 2y}$. The steel beams in the first tier, being considered as cantilevers with a projection equal to y and having a center of moments about the line $a\,b$, sustain the uniformly distributed load acting upwards from beneath, equal to the load per lineal foot of the tier multiplied by the distance y, or, as it may be stated, $\frac{Wy}{x + 2y}$. The center of gravity of this uniformly distributed load is along the line $c\,d$, the distance of $c\,d$ from $a\,b$ being $\frac{y}{2}$; therefore, the bending moment of all the beams in the first tier is equal to the moment of the uniformly distributed load on the projecting portion of the tier between the edge of the cast-iron base and the ends of the beams, or, algebraically expressed,

$$M = \frac{Wy^2}{2(x + 2y)} \qquad (1)$$

It is usual to take all the lengths in inches and the weight in pounds, for then the bending moment M will be in inch-pounds.

This formula, based on the assumption that the steel beams are cantilevers, may be stated as in the following rule:

Rule.—*The bending moment on the beams in any tier is equal to the quotient obtained by dividing the product of the load on the footings and the square of the distance that the beams project beyond the base or the tier of beams above, by twice the sum obtained by adding the width of the tier or base above and twice the projection.*

8. The resisting moment in any steel beam must equal the bending moment; hence, $M = M_1$, and $M_1 = Ss$, when S equals the section modulus of the beam section, and s the

ultimate unit fiber stress. When the bending moment has been determined and the number of beams in the tier decided on, the required section modulus for each beam may be obtained by the formula

$$S = \frac{M_1}{s_a n} \qquad (2)$$

in which M_1 = resisting moment;
s_a = allowable unit fiber stress;
n = number of steel beams in tier.

This formula may be expressed as follows:

Rule.—*The section modulus required for each steel beam in a grillage footing is determined by dividing the resisting moment necessary for the entire tier by the product of the safe unit fiber stress and the number of beams.*

On obtaining the necessary section modulus for each of the steel beams in this manner the most economical beam section may be determined from tables giving properties of sections.

Instead of assuming the number of beams in the tier and using formula 2, to find the required section modulus, the size of the beam could have been decided on and the number required obtained by transposing the formula thus,

$$n = \frac{M_1}{s_a S} \qquad (3)$$

If the number of beams determined in this manner is too great to be placed under the cast-iron cap and allow at least 3 inches between the flanges, then either the cap must be increased in width or deeper and heavier beams must be adopted. After the first tier of beams has been designed and their size determined by formula 2, the size of the beams for the second tier, that is, the tier beneath, may be computed.

EXAMPLE.—In Fig. 6, it is assumed that the dimensions of the footings are as shown and that the number of beams in each tier and the dimensions of the cast-iron base have been designed as shown on the plan. What should be the size of the steel beams in each tier in order to provide ample support for the load of 400,000 pounds, using a safe unit fiber stress of 15,000 pounds?

SOLUTION.—The bending moment on the upper tier of beams a, a is, by formula **1**, equal to M, or $\frac{W y^2}{2(x+2y)}$. According to the conditions of the problem and the dimensions given on the figure, $W = 400,000$ lb., $y = 56$ in., and $x = 30$ in. Then, by substitution, $M = \frac{400,000 \times 56 \times 56}{2(30 + 2 \times 56)} = 4,416,901$ in.-lb. From formula **2**, the required section modulus is equal to $S = \frac{M_1}{s_a n}$. Since $M_1 = M$, the value of M_1 in the problem equals the result just obtained, or 4,416,901 in.-lb; s_a taken at 15,000 lb. gives a factor of safety of at least 4, which is ample, and n, the number of beams taken from the figure,

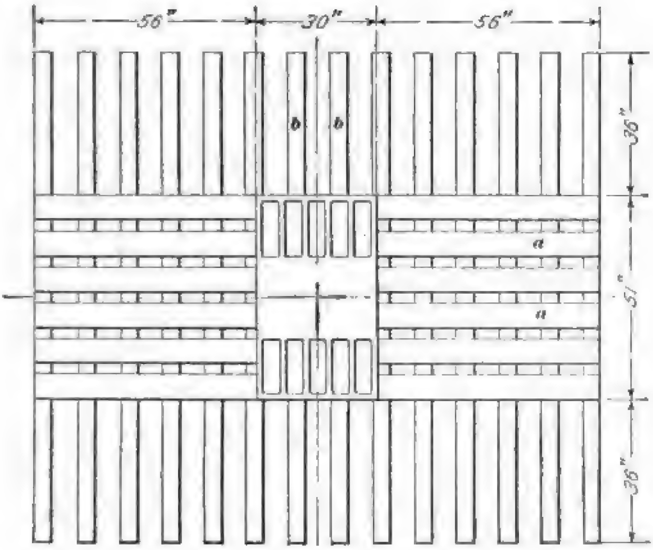


FIG. 6

equals 6, so that by substituting these values in the formula, $S = \frac{4,416,901}{15,000 \times 6} = 49.07$, which is the section modulus required for each beam of the first tier. From tables giving the properties of sections, the most economical I beam will be found to be one having a depth of 15 in. and a weight of 42 lb. per ft. By substituting in formula **1**, the bending moment on the beams b, b in the second tier is equal to $M = \frac{400,000 \times 36 \times 36}{2(51 + 2 \times 36)} = 2,107,317$ in.-lb. Then, since M_1 has been determined and the safe unit fiber stress is 15,000 lb., and there are

fourteen beams, the value of S, from formula **2**, is equal to $\frac{2,107,317}{15,000 \times 14}$ = 10.035. From the tables, it will be observed that beams having a depth of 7 in. and weighing 15 lb. per ft. may be used in the bottom tier. Ans.

9. The other method for determining the transverse resistance of the steel beams in the grillage foundation is

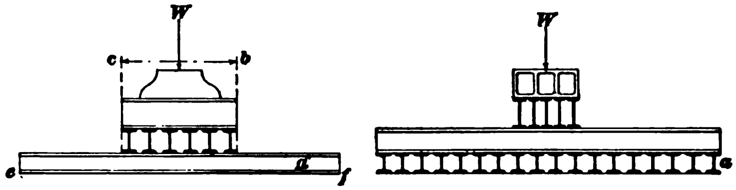


FIG. 7

more reasonable, and preferable to the one just explained. In this method, the entire length of the beam is considered in calculating the bending moment and the loads on the beams are taken as they actually exist.

For instance, in Fig. 7 is shown a steel-beam grillage, in elevation, composed of three tiers of beams. In analyzing the bending moment on the lower tier a, it will be seen that the total load acting on the top of the beam is equal to the entire load from the column distributed over the area $c\,b$ in length, while the reactions acting upwards from the bottom of the beam in opposition to the weight from the column are distributed over an area equal in length to $e\,f$, the entire reaction being, of course, equal to the load W. The condition of loading that then exists on the beam is diagrammatically shown in Fig. 8, in which the load on the top of the beam is equal in amount to the force acting upwards from the bottom. The greatest bending moment under such a condition of loading does not actually occur at the edge of the first tier of beams g, as explained in connection with

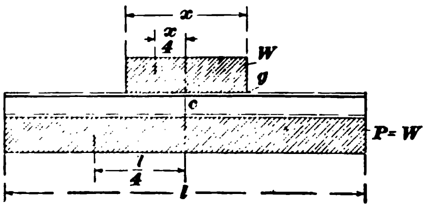


FIG. 8

formula **1**, but exists at the center c of the beam, Fig. 8. Then, considering the point c as the center of moments, the bending moment at that point will be equal to the difference between the moment of the force acting upwards and that of the load on the beam acting downwards, which is according to the principle of moments.

The lever arm of the upward force, or the reaction, is equal to $\frac{l}{4}$ and that of the downward force is $\frac{x}{4}$, or the distances from the center of moments to the centers of gravity of one-half the respective loads. The positive moment is equal to $\frac{W}{2} \times \frac{l}{4} = \frac{Wl}{8}$ and the negative moment is equal to $\frac{W}{2} \times \frac{x}{4} = \frac{Wx}{8}$; then the bending moment $M = \frac{Wl}{8} - \frac{Wx}{8}$,

or
$$M = \frac{W}{8}(l - x) \qquad (4)$$

The method for obtaining the bending moment as explained by the above formula may be stated as follows:

Rule.—*The bending moment on any tier of steel beams in the grillage footing is equal to one-eighth of the entire weight on the column, multiplied by the difference between the width of the base plate or upper tier of beams and the length of the beams in the tier beneath.*

EXAMPLE.—According to formula **4**, what size beams will be required in the steel-beam grillage footing shown in Fig. 9, considering a safe unit fiber stress of 15,000 pounds, provided that the load, as in the previous example, is equal to 400,000 pounds?

SOLUTION.—In the upper tier of beams, the distance l in this case is equal to their length, or 142 in., while the distance x is equal to the width of the base plate, or 30 in. By substituting these values, together with the total load W, or 400,000 lb., in formula **4**, $M = \frac{400,000}{8}(142 - 30) = 5,600,000$ in.-lb. The bending moment on the beams in the second tier by the same formula is equal to $M = \frac{400,000}{8}(123 - 51) = 3,600,000$ in.-lb. The section modulus for each beam may then be determined by applying formula **2**, or $S = \frac{M_1}{s_a n}$. On substituting the respective values in this formula, the

section modulus for the upper tier is equal to $\frac{5,600,000}{15,000 \times 6} = 62.22$, while for the lower tier the section modulus equals $\frac{3,600,000}{15,000 \times 14} = 17.14$. From the table in *Properties of Sections*, a 15-in. I beam weighing 50 lb. will be found most economical for the upper tier, while for the lower tier a 9-in. I beam weighing 21 lb. is suitable. Ans.

It will be observed that by this method of calculation, heavier beams are required and the formula consequently gives safer results. The principles on which the formula is

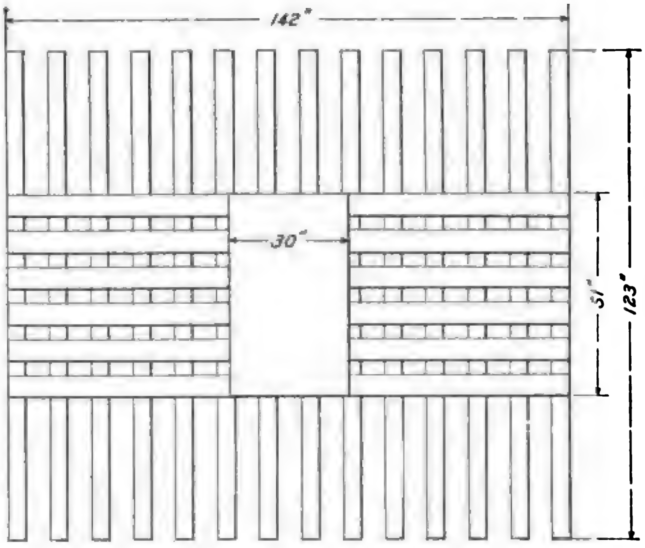


FIG. 9

based are theoretically correct, whereas, in the other method they were not, so that the more conservative rule should be used. Both methods are given here that the student may know the general practice in the design of these footings.

10. For convenience in figuring the strength of steel beams in grillage footings, the following table is given. This table gives the safe load on a single beam in tons of 2,000 pounds when a safe unit fiber stress of 16,000 pounds is assumed. In applying the table, it is necessary to determine

TABLE I

SAFE LOAD ON ONE BEAM, IN TONS OF 2,000 POUNDS

Beam		Unloaded Length of Beam, $l-x$ in Inches												
Depth in Inches	Weight in Pounds per Foot	36	48	60	72	84	96	108	120	132	144	156	168	180
24	100					151.1	132.2	117.5	105.7	96.1	88.1	81.3	75.5	70.5
24	95					146.6	128.3	114.0	102.6	93.3	85.5	78.9	73.3	68.4
24	90					142.1	124.3	110.5	99.5	90.4	82.9	76.5	71.1	66.3
24	85					137.7	120.4	107.1	96.4	87.6	80.3	74.1	68.8	64.2
24	80					132.5	115.9	103.1	92.6	84.3	77.3	71.3	66.3	61.8
20	75				112.8	96.9	84.6	75.2	67.7	61.4	56.4	52.1	48.3	45.1
20	70				108.4	93.0	81.3	72.3	65.1	59.2	54.2	50.1	46.5	43.4
20	65				104.0	89.1	78.0	69.3	62.4	56.7	52.0	48.0	44.6	41.6
18	70			109.2	91.0	78.0	68.3	60.7	54.6	49.7	45.5	42.0	39.0	36.4
18	65			104.4	87.0	74.6	65.3	58.0	52.2	47.5	43.5	40.2	37.3	34.8
18	60			99.7	83.1	71.2	62.3	55.4	49.9	45.3	41.6	38.4	35.6	33.2
18	55			94.3	78.6	67.4	58.9	52.4	47.1	42.9	39.3	36.3	33.7	31.4
15	60		95.7	76.7	63.8	54.7	47.9	42.6	38.3	34.8	31.9	29.5	27.4	25.5
15	55		90.8	72.6	60.5	51.9	45.4	40.4	36.3	33.2	30.3	27.9	25.9	24.2
15	50		86.0	68.8	57.3	49.1	43.0	38.2	34.4	31.3	28.7	26.5	24.6	22.9
15	45		81.1	64.9	54.0	46.3	40.5	36.0	32.4	29.5	27.0	24.9	23.2	21.6
15	42		78.5	62.8	52.4	44.9	39.3	34.9	31.4	28.6	26.2	24.2	22.4	20.9
12	40	72.9	54.7	43.7	36.4	31.2	27.3	24.3	21.9	19.9	18.2	16.8	15.6	14.6
12	35	67.6	50.7	40.5	33.8	29.0	25.3	22.5	20.2	18.4	16.9	15.6	14.5	13.5
12	31½	64.0	48.0	38.4	32.0	27.4	24.0	21.3	19.2	17.4	16.0	14.8	13.7	12.8
10	40	56.4	42.3	33.8	28.2	24.2	21.1	18.8	16.9	15.4	14.1	13.0	12.1	11.3
10	35	52.1	39.1	31.3	26.0	22.3	19.5	17.4	15.6	14.2	13.0	12.0	11.2	10.4
10	30	47.6	35.7	28.6	23.8	20.4	17.9	15.9	14.3	13.0	11.9	11.0	10.2	9.5
10	25	43.4	32.5	26.0	21.7	18.6	16.3	14.5	13.0	11.8	10.8	10.0	9.3	8.7
9	35	44.1	33.1	26.5	22.0	18.9	16.5	14.7	13.2	12.0	11.0	10.2	9.5	8.8
9	30	40.2	30.1	24.1	20.1	17.2	15.1	13.4	12.1	11.0	10.0	9.1	8.6	8.0
9	25	36.3	27.2	21.8	18.1	15.5	13.6	12.1	10.9	9.9	9.1	8.4	7.8	7.3
9	21	33.6	25.2	20.2	16.8	14.4	12.6	11.2	10.1	9.2	8.4	7.8	7.2	6.7
8	25¼	30.2	22.7	18.1	15.1	13.0	11.3	10.1	9.1	8.2	7.6	7.0	6.5	6.0
8	22¾	28.4	21.3	17.1	14.2	12.2	10.7	9.5	8.5	7.8	7.1	6.6	6.1	5.7
8	20¼	26.7	20.0	16.0	13.3	11.4	10.0	8.9	8.0	7.3	6.7	6.2	5.7	5.3
8	18	25.3	18.9	15.2	12.6	10.8	9.5	8.4	7.6	6.9	6.3	5.8	5.4	5.1
7	20	21.5	16.1	12.9	10.8	9.2	8.1	7.2	6.5	5.9	5.4	5.0	4.6	4.3
7	17½	19.9	14.9	12.0	10.0	8.5	7.5	6.6	6.0	5.4	5.0	4.6	4.3	4.0
7	15	18.5	13.9	11.1	9.2	7.9	6.9	6.2	5.6	5.0	4.6	4.3	4.0	3.7
6	17¼	15.5	11.6	9.3	7.7	6.6	5.8	5.2	4.6	4.2	3.9	3.6	3.3	3.1
6	14¾	14.2	10.7	8.5	7.1	6.1	5.3	4.7	4.3	3.9	3.6	3.3	3.0	2.8
6	12¼	13.0	9.7	7.8	6.5	5.6	4.9	4.3	3.9	3.5	3.2	3.0	2.8	2.6
5	14¾	10.8	8.1	6.5	5.4	4.7	4.1	3.6	3.3	3.0	2.7	2.5		
5	12¼	9.6	7.2	5.8	4.8	4.1	3.6	3.2	2.9	2.6	2.4	2.2		
5	9¾	8.5	6.4	5.1	4.3	3.7	3.2	2.8	2.6	2.3	2.1	2.0		
4	10½	6.4	4.8	3.8	3.2	2.7	2.4	2.1						
4	9½	6.0	4.5	3.6	3.0	2.6	2.3	2.0						
4	8½	5.7	4.3	3.4	2.8	2.4	2.1	1.9						
4	7½	5.3	4.0	3.2	2.7	2.3	2.0	1.8						
3	7½	3.4	2.5	2.0										
3	6½	3.2	2.4	2.0										
3	5½	3.0	2.3	1.8										

the value of $l - x$, or the difference between the length of the steel beams beneath and the width of the steel-beam grillage, or the cast-iron base above, as shown in Fig. 10. It is also necessary to determine the uniform load on each beam in the tier under consideration, which has been found by dividing the total load on the footing by the number of beams. In the column giving the existing value of $l - x$, select the value nearest to the uniformly distributed load, in

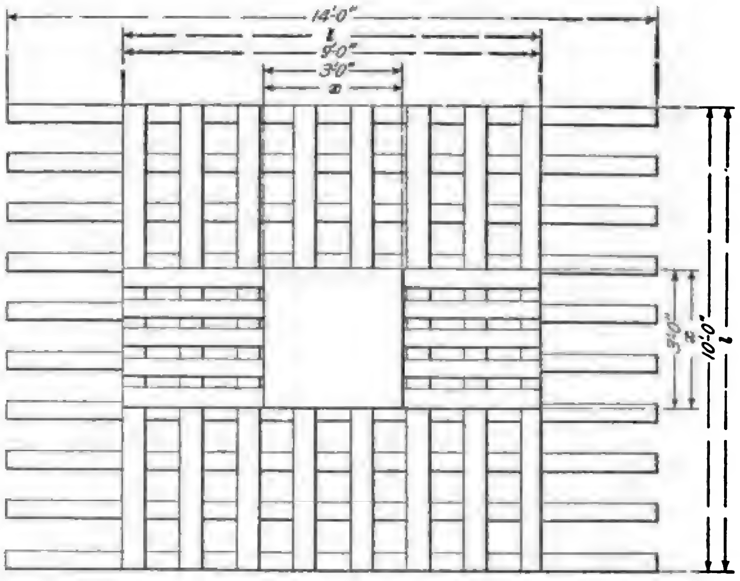


FIG. 10

tons, on the beam and by referring to the columns headed Depth and Weight, the size of the steel beam for the tier will be determined.

For example, Fig. 10 shows a diagrammatic plan of a three-tier, steel-beam, grillage footing. The load on the column is 350,000 pounds, or 175 tons, so that the load on each beam in the several tiers is as follows:

$$\begin{aligned} \text{Top tier} &= 175 \div 5 = 35 \text{ tons} \\ \text{Middle tier} &= 175 \div 8 = 21.875 \text{ tons} \\ \text{Bottom tier} &= 175 \div 10 = 17.5 \text{ tons} \end{aligned}$$

The values of $l - x$ for the several tiers may be determined thus:

Top tier $= 9 - 3 = 6$ ft. $= 72$ in.
Middle tier $= 10 - 3 = 7$ ft. $= 84$ in.
Bottom tier $= 14 - 9 = 5$ ft. $= 60$ in.

Then, referring to Table I, for the first tier of beams, under the column headed 72, it is found that a 12-inch 40-pound beam is the lightest that will support the required load and that for the middle tier of beams, under the column headed 84, a 12-inch $31\frac{1}{2}$-pound beam will be the most economical, while for the bottom tier of beams, under the column headed 60, the most economical beam will be the 9-inch 21-pound beam.

In calculating the values given in this table, a formula that has been evolved from the elementary equation $M = M_1$ has been used. M_1 equals the resisting moment or Ss, while M equals the bending moment which, according to formula 4, is $\frac{W}{8}(l - x)$. Substituting these values in the equation $M = M_1$ gives $Ss = \frac{W}{8}(l - x)$. Dividing through by W gives $\frac{Ss}{W} = \frac{W(l - x)}{8\,W}$, and by cancelation the equation becomes $\frac{Ss}{W} = \frac{l - x}{8}$. In this equation, s is the ultimate unit fiber stress, and by dividing by the factor of safety the safe unit stress is obtained, which, according to the table, is equal to 16,000 pounds. Substituting this value, the formula becomes $\frac{16{,}000\,S}{W} = \frac{l - x}{8}$. W is in pounds while, according to the table, it should be in tons, so by dividing the left-hand member of the equation by 2,000, the formula is changed to $\frac{8\,S}{W} = \frac{l - x}{8}$; by transposition, $W = \frac{64\,S}{l - x}$, which is the formula by which the values in the table are calculated.

EXAMPLES FOR PRACTICE

1. A grillage footing is composed of two tiers of steel beams; the column base, which is 30 inches wide, supports a load of 275,000 pounds. The steel beams in the first tier are five in number and the tier is 42 inches in width, measuring between the outside edges of the flanges of the outside beams, and the length of the beams in this tier is 150 inches. The number of beams in the bottom tier is fifteen and their length is also 150 inches. Provided that an allowable unit fiber stress of 18,000 pounds is assumed and formula 4 is employed, what will be the economical sizes of the beams in both tiers?

Ans. { Upper tier, 15-in. 45-lb. beams
Lower tier, 9-in. 21-lb. beams

2. What will be the sizes of the beams required in problem 1 if formula 1 is used?

Ans. { Top tier, 12-in. 35-lb. beams
Bottom tier, 7-in. 15-lb. beams

3. Determine, by means of Table I, the economic sizes of steel beams required for a grillage footing composed of three tiers of steel beams. The bottom tier is 12 feet 6 inches long and 10 feet wide, the intermediate tier has a length equal to the width of the bottom tier and a width of 6 feet, while the top tier extends across the intermediate tier and is 4 feet 6 inches wide. The cast-iron base is rectangular in plan, being equal in length to the width of the top tier and 3 feet wide. Six beams compose the top tier, while in the intermediate and bottom tiers there are nine and twelve, respectively. The load on the footing is from a principal column and amounts to 320 tons.

Ans. { Bottom tier, 12-in. 31½-lb. beams
Intermediate tier, 12-in. 40-lb. beams
Top tier, 12-in. 31½-lb. beams

NOTE.—When the required value of $l - x$ is not given in the table, the value next higher should be taken.

FOOTINGS SUPPORTING TWO COLUMNS

11. In the design of grillage foundations it is often required that one system of grillage will support two columns, as shown in Fig. 11. In such cases the greatest bending moment may occur on the beams *d* at either the point of no shear *c* or beneath one of the columns. The distance of the point of no shear *c* from either end of the tier may be determined by calculating the unit pressure from the soil and dividing the load supported by the column near the end of the tier in question by this unit pressure.

If the column bases or the first tiers of beams supporting

the bases are considered as concentrated loads on the lower tier of beams at single points *b*, *b* and the pressure from the soil is assumed to be equally distributed throughout the length of the footing, the greatest bending moment, either at the points *b*, *b* or at the center *c* may be obtained from the formula given for this condition of loading in

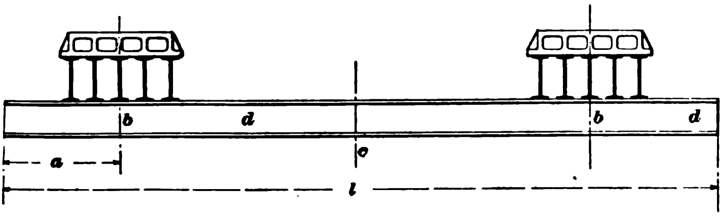


FIG. 11

Beams and Girders, Part 1. It will also be observed that the bending moment at the center and the one under the concentrated load are practically equal when the ratio of the distances from the ends of the beams to the concentrated loads are to the entire length of the beam as .207 is to 1, or, referring to Fig. 11, when the distance *a* is .207 of *l*. This formula, however, will give a bending moment greater than

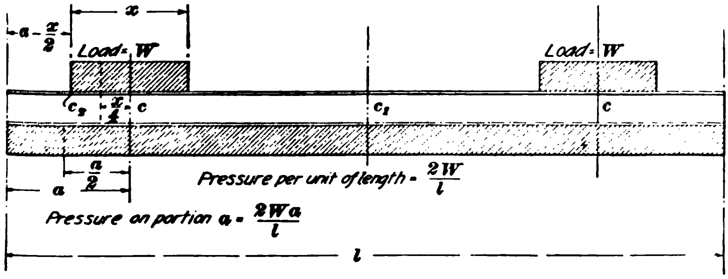


FIG. 12

actually exists, from the fact that the load from the cast-iron base is not concentrated as assumed, but is distributed over a portion of the length of the grillage. The actual conditions of loading are shown in Fig. 12, and the grillage beams should be analyzed for the bending moment, under the center of the column, as at *c*; at the point of no shear, as

at c_1; and at the point c_2, located at the outside edge of the steel beams in the upper tier, or of the cast-iron base, if it sets directly on the beams under consideration.

The formula for the bending moment about the point c, provided that W represents the load on the columns, may be obtained from the principle of moments, as follows: The negative moment about the point c acting in a downward direction is equal to $\frac{W}{2} \times \frac{x}{4} = \frac{Wx}{8}$. The positive moment acting upwards in opposition to the moment just found is equal to the pressure from the soil from the end of the beam to the point c, multiplied by one-half of this distance. The load from the soil on this portion of the beam is equal to the pressure per unit of length on the beam multiplied by the distance from the end to the point c. The pressure on each unit of length is equal to the total load divided by the length, or $\frac{2W}{l}$, and the entire force on this portion of the beam is equal to $\frac{2Wa}{l}$. The moment of this force about c is equal to $\frac{2Wa}{l} \times \frac{a}{2} = \frac{2Wa^2}{2l} = \frac{Wa^2}{l}$. From this expression, which represents the positive moment, is subtracted the negative moment, which, as found, is equal to $\frac{Wx}{8}$; hence, the expression for the bending moment is

$$M = \frac{Wa^2}{l} - \frac{Wx}{8} = \frac{W}{8}\left(\frac{8a^2}{l} - x\right)$$

The bending moment about c_1 is equal to the difference between the positive and negative moments. The negative moment about this point is equal to $W\left(\frac{l}{2} - a\right)$, while the positive moment is equal to $\frac{Wl}{4}$. The algebraic sum of these moments equals $\frac{Wl}{4} - W\left(\frac{l}{2} - a\right)$, or $W\left(a - \frac{l}{4}\right)$; consequently, the bending moment M at the center of the beam equals $W\left(a - \frac{l}{4}\right)$.

The bending moment about the point c_2 is also to be determined. There is but one moment, an upward one, about this point, which is due to the pressure of the soil on the projecting length of the steel beam, and therefore the end of the steel beam may be regarded as a cantilever with a uniformly distributed load equal to $\frac{2W\left(a-\frac{x}{2}\right)}{l}$.

The lever arm of this load is equal to $\frac{a-\frac{x}{2}}{2}$ and the moment about c_2 equals $\frac{2W\left(a-\frac{x}{2}\right)}{l} \times \frac{\left(a-\frac{x}{2}\right)}{2}$, so that the bending moment M about the point c_2 equals $\frac{W\left(a-\frac{x}{2}\right)^2}{l}$.

It will be necessary, therefore, in the design of any grillage footing supporting two columns in the manner shown in Fig. 11, to calculate these three bending moments in order to determine which one is the greatest so that the beams may be proportioned for the maximum moment.

12. The method of procedure for obtaining the maximum bending moment on such a footing may be expressed by the following rule:

Rule.—*Calculate the bending moments beneath the central axes of the columns, at the edge of the superimposed tier of beams, and at the center of the tier under consideration. By inspection, select the maximum bending moment and proportion the steel beams accordingly.*

The bending moments are equal to the values obtained by the following formulas:

The bending moment beneath the central axis of the column equals Mc.

$$Mc = \frac{W}{8}\left(\frac{8a^2}{l} - x\right) \qquad (5)$$

The bending moment at the center of the tier of beams under consideration equals Mc_1.

$$Mc_1 = W\left(a - \frac{l}{4}\right) \qquad (6)$$

The bending moment at the edge of the superimposed tier of beams equals Mc_2.

$$Mc_2 = \frac{W\left(a - \frac{x}{2}\right)^2}{l} \qquad (7)$$

in which W = load on one column;
l = length of tier of beams under consideration;
x = width of superimposed tier of beams;
a = distance from end of tier of beams in question to central axis of column. As these distances are taken in inches or feet, the results will be in inch-pounds or foot-pounds.

The bending moment at the center of the tier of beams is zero when the distance a, Fig. 13, exceeds one-fourth of l, the length of the tier. When a is less than $\frac{l}{4}$, the bending moment at the center is negative, or opposite in effect to

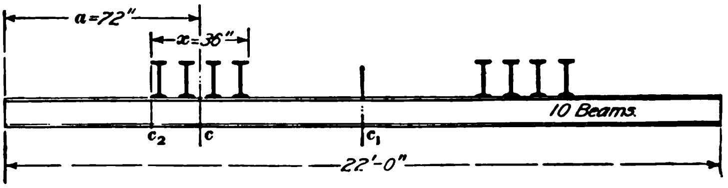


FIG. 13

that created under the columns or at the edge of the superimposed tier of beams.

EXAMPLE.—Determine the most economical I beams that may be used for the bottom tier of the grillage footing shown in Fig. 13, provided that there are ten beams in the tier and that a safe unit stress of 18,000 pounds is assumed, the load from each superimposed column being 375,000 pounds.

SOLUTION.—It is necessary to determine the bending moment at the three points, c, c_1, and c_2, which may be accomplished by applying

formulas **5**, **6**, and **7**, respectively. The bending moment at c, according to formula **5**, is equal to $\frac{W}{8}\left(\frac{8a^2}{l} - x\right)$, and by substitution is equal to $\frac{375,000}{8}\left(\frac{8 \times 72^2}{264} - 36\right) = 5,676,094$ in.-lb. The bending moment about the center c_1 of the steel-beam grillage in the bottom tier, according to formula **6**, is equal to $W\left(a - \frac{l}{4}\right)$, which, by substitution, equals $375,000\left(72 - \frac{264}{4}\right) = 2,250,000$ in.-lb. By making similar calculations for the bending moment about c_2, using formula **7**, or $W\frac{\left(a - \frac{x}{2}\right)^2}{l}$, the value, by substitution, is found to equal $\frac{375,000 \times (72 - \frac{36}{2})^2}{264} = 4,142,045$ in.-lb. The bending moment at the several points may then be tabulated as follows:

At c, 5,676,094 in.-lb.
At c_1, 2,250,000 in.-lb.
At c_2, 4,142,045 in.-lb.

The greatest bending moment in this particular case exists under the center of the column or at the point c. Since there are ten beams in the tier, the bending moment on each beam will equal 5,676,094 ÷ 10 = 567,609 in.-lb. The required section modulus will equal the bending moment on each beam divided by the safe unit fiber stress which, according to the problem, is 18,000 lb., so that the section modulus in this instance is 567,609 ÷ 18,000 = 31.53. From tables giving the properties of I beams it will be found that a 12 -in. I beam weighing 31½ lb. per ft. has a section modulus of 36, and that this is the least size beam that will give the required resistance. Ans.

EXAMPLES FOR PRACTICE

1. Determine the size of the steel beams necessary in the bottom tier of a grillage foundation to support two columns, each carrying 200 tons. The distance between the centers of the columns is 2 feet and the distance from the center of each column to the end of the tier is 6 feet 6 inches. The upper tier of beams is 4 feet wide and the number of beams in the bottom grillage is 12. Assume a safe unit fiber stress of 15,000 pounds. Ans. 15-in. 50-lb. beams

2. Two columns supporting loads of 310 tons each rest on a tier of steel beams that are supported on a bottom tier. The distance from center to center of columns is 12 feet and the distance from the center of columns to the end of the under tier is in this case 5 feet. Provided that the width of the upper tier between the outside flange edges

is 40 inches and an allowable unit fiber stress of 16,000 pounds is assumed, what will be the size of the steel beams in the lower tier if ten beams are used? Ans. 18-in. 55-lb. beams

---

13. When a continuous foundation supports two columns carrying unequal loads, the footing is made in the shape of a trapezoid instead of a rectangle. This is done so that the unit pressure from the soil beneath may be as nearly uniform as possible. The method for determining the greatest bending moment on such a footing is explained in Art. **27.**

FOOTINGS SUPPORTING THREE COLUMNS

14. Necessity sometimes arises for placing three-column footings on a continuous steel-beam grillage, as is shown in Fig. 14. In this case it is necessary to support the extreme exterior column on a cantilever, as shown at *a*. This means was resorted to in order that the footings of the old building wall need not be disturbed. The load on the exterior column is transmitted through a cantilever composed of a plate or box girder secured to a basement column, as at *c*, well within the building line. In such a footing, the center of gravity of the combined area must coincide with the center of gravity of the loads, and the columns must be so placed and loaded as to accomplish this, or the footings must be spread at one end and narrowed at the other so that the base is a trapezoid, in which case the center of gravity of the area will be moved to approach approximately the center of gravity of the loads. Should it be impossible to so proportion these footings that the center of gravity of the load coincides with the center of gravity of the footing area, independent grillage footings must be employed, or this type of foundation abandoned and other means resorted to, such as piling or piers built by sinking caissons.

15. In figuring the bending stresses on a grillage where three columns are supported on a single tier, the formula for continuous beams given in *Beams and Girders*, Part **1**, may be employed. But from the fact that the beams used

in such a foundation will be of considerable depth, and also because they are supported on a base of concrete of considerable thickness, besides being thoroughly embedded in the concrete, a great deflection downwards under the columns and upwards between them cannot occur. Therefore, the pressure on the soil will not vary at different points throughout the length of the footing, but may be assumed as practically uniform. On such a basis of figuring, the analysis of the bending stresses may be made.

In proceeding with the analysis of the bending stresses in a three-column footing, it is first necessary to determine the points at which the shear changes sign, or where it equals zero, for at these points the bending moments are greatest. The

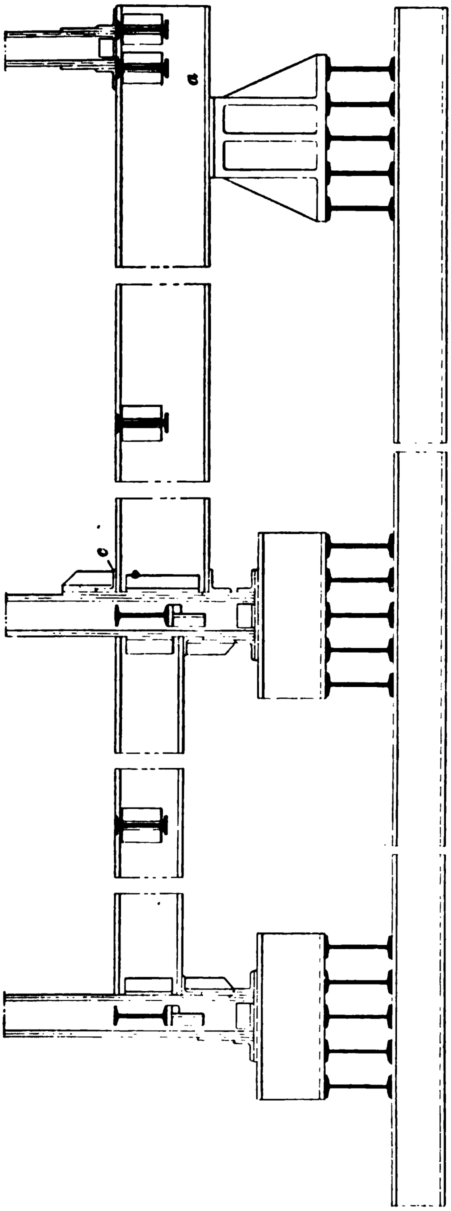
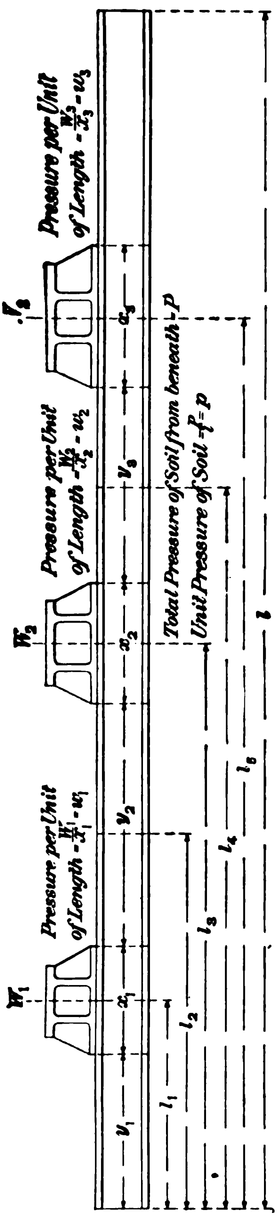


FIG. 14

points of no shear may be readily ascertained by taking the algebraic sum of the loads throughout the beam; for instance, in Fig. 15 is designated a steel-beam grillage supporting three columns. The total load from the columns is represented by W_1, W_2, and W_3, while the total pressure from the soil beneath is designated by P. Since the sum of the loads must equal the sum of the reactions in order that any beam may be in equilibrium, the following expression must be true: $P = W_1 + W_2 + W_3$.

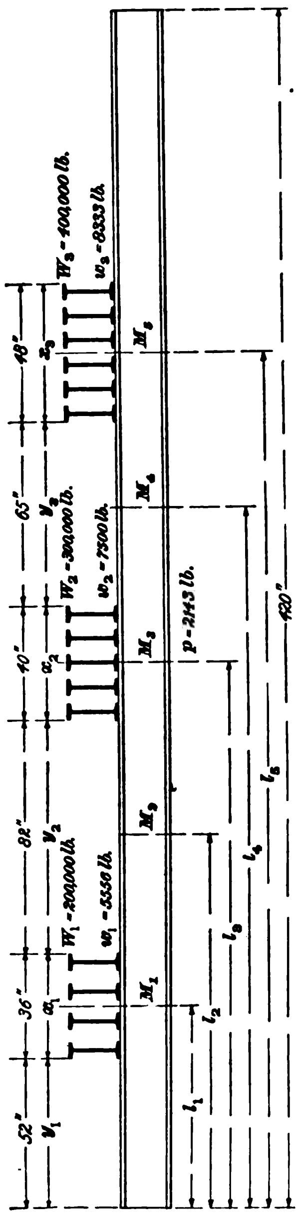
The unit load on the steel-beam grillage from the pressure of the soil beneath is equal to $\frac{P}{l}$; this value may be represented by p. The pressure per unit of length from the columns, either supported on the cross-grillage or on cast-iron bases, may be represented by w_1, w_2, w_3, respectively, and these values are found by the equations $w = \frac{W_1}{x_1}$, $w_2 = \frac{W_2}{x_2}$, and $w_3 = \frac{W_3}{x_3}$. It is the purpose to find the lengths l_1, l_2, l_3, l_4, and l_5, which are the distances from the left-hand end of the grillage to the several points at which the shear changes sign or equals zero. The formulas for these several distances may be worked out from the principle, which has for its foundation the fact that the shear at any point is equal to the algebraic sum of the loads at that point.

The formulas that will give the required distances from



Pressure per Unit of Length = $\frac{W_1}{x_1} = w_1$
Pressure per Unit of Length = $\frac{W_2}{x_2} = w_2$
Pressure per Unit of Length = $\frac{W_3}{x_3} = w_3$
Total Pressure of Soil from beneath = P
Unit Pressure of Soil = $\frac{P}{l} = p$

FIG. 15



$W_1 = 200,000$ lb.
$w_1 = 5556$ lb.
$W_2 = 300,000$ lb.
$w_2 = 7500$ lb.
$W_3 = 400,000$ lb.
$w_3 = 8333$ lb.
$p = 2143$ lb.
52"
36"
82"
40"
65"
48"
420"

FIG. 16

points of no shear may be readily ascertained by taking the algebraic sum of the loads throughout the beam; for instance, in Fig. 15 is designated a steel-beam grillage supporting three columns. The total load from the columns is represented by W_1, W_2, and W_3, while the total pressure from the soil beneath is designated by P. Since the sum of the loads must equal the sum of the reactions in order that any beam may be in equilibrium, the following expression must be true: $P = W_1 + W_2 + W_3$.

The unit load on the steel-beam grillage from the pressure of the soil beneath is equal to $\frac{P}{l}$; this value may be represented by p. The pressure per unit of length from the columns, either supported on the cross-grillage or on cast-iron bases, may be represented by w_1, w_2, w_3, respectively, and these values are found by the equations $w = \frac{W_1}{x_1}$, $w_2 = \frac{W_2}{x_2}$, and $w_3 = \frac{W_3}{x_3}$. It is the purpose to find the lengths l_1, l_2, l_3, l_4, and l_5, which are the distances from the left-hand end of the grillage to the several points at which the shear changes sign or equals zero. The formulas for these several distances may be worked out from the principle, which has for its foundation the fact that the shear at any point is equal to the algebraic sum of the loads at that point.

The formulas that will give the required distances from the left-hand end of the grillage, using the notation designated in Fig. 15, are as follows:

$$l_1 p = w_1(l_1 - y_1), \text{ or } l_1 = \frac{y_1 w_1}{w_1 - p} \qquad (8)$$

$$l_2 p = W_1, \text{ or } l_2 = \frac{W_1}{p} \qquad (9)$$

$$l_3 p = W_1 + [l_3 - (y_1 + x_1 + y_2)] w_2$$

or

$$l_3 = \frac{W_1 - (y_1 + x_1 + y_2) w_2}{p - w_2} \qquad (10)$$

$$l_4 p = W_1 + W_2, \text{ or } l_4 = \frac{W_1 + W_2}{p} \qquad (11)$$

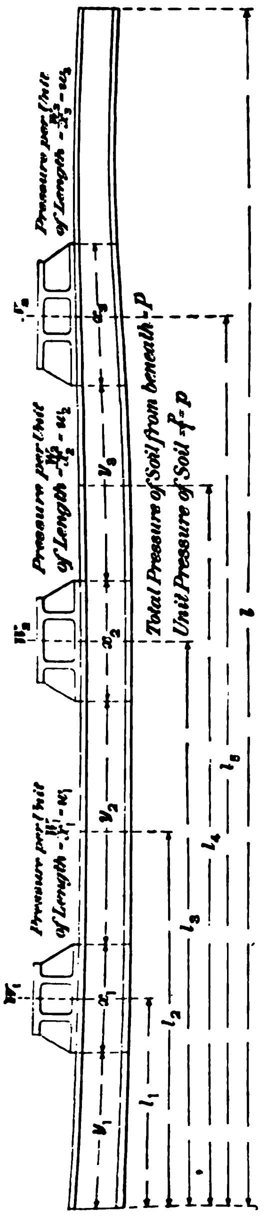

Pressure per Unit of Length = $\frac{W_1}{x_1} = w_1$
Pressure per Unit of Length = $\frac{W_2}{x_2} = w_2$
Pressure per Unit of Length = $\frac{W_3}{x_3} = w_3$
Total Pressure of Soil from beneath = P
Unit Pressure of Soil = $\frac{P}{l} = p$

FIG. 15

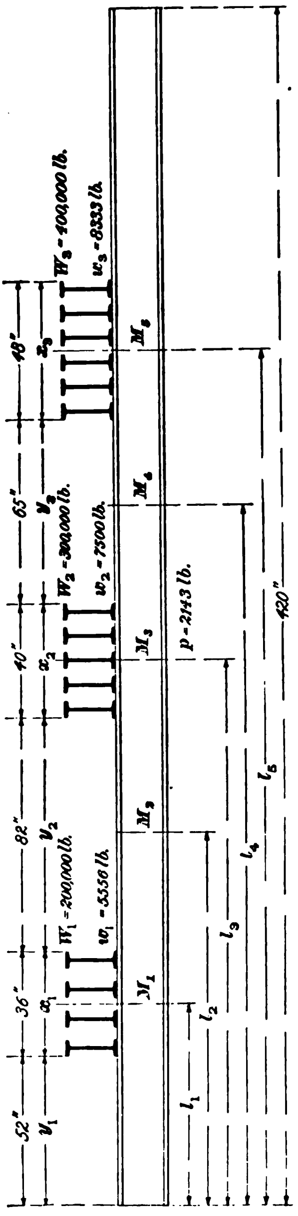

W_1 = 200,000 lb.
w_1 = 5556 lb.
W_2 = 300,000 lb.
w_2 = 7500 lb.
W_3 = 400,000 lb.
w_3 = 8333 lb.
p = 2143 lb.
52″
36″
82″
40″
65″
48″
120″

FIG. 16

EXAMPLES FOR PRACTICE

1. Determine the points of no shear, or the points at which the greatest bending moments occur in the grillage shown in Fig. 17 when W_1 equals 300,000 pounds, W_2 equals 250,000 pounds, W_3 equals 325,000 pounds, while the distances y_1, x_1, y_2, x_2, y_3, and x_3 equal, respectively, 58, 40, 80, 36, 72, 48, and the length of the grillage equals 33 feet.

$$\text{Ans.}\begin{cases} l_1 = 82.23 \text{ in.} = 6 \text{ ft. } 10\tfrac{1}{4} \text{ in.} \\ l_2 = 135.75 \text{ in.} = 11 \text{ ft. } 3\tfrac{3}{4} \text{ in.} \\ l_3 = 197.73 \text{ in.} = 16 \text{ ft. } 5\tfrac{3}{4} \text{ in.} \\ l_4 = 248.87 \text{ in.} = 20 \text{ ft. } 8\tfrac{7}{8} \text{ in.} \\ l_5 = 303.99 \text{ in.} = 25 \text{ ft. } 4 \text{ in.} \end{cases}$$

2. If the values of y_1, x_1, y_2, x_2, y_3, and x_3 in Fig. 17 are, respectively, equal to 60, 40, 90, 48, 80, and 44 inches, and the length of the bottom tier of beams is 35 feet 4 inches, while the loads W_1, W_2, and W_3 are equal to 150, 200, and 175 tons, in the order named, what will be

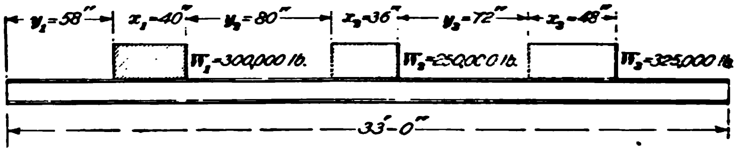


FIG. 17

the size beams required in the lower tier, provided that they are spaced 12 inches from center to center? The footing is 11 feet wide between the centers of outside beams, and the safe unit fiber stress is 18,000 pounds. Ans. 20-in. 75-lb. beams

REENFORCED CONCRETE FOOTINGS

17. The **reenforced concrete footing**, a type of which is shown in Fig. 18, is meeting with considerable favor among engineers in the construction of spread footings for the support of high buildings. A footing built of reenforced concrete possesses all of the advantages of a grillage footing, and its cost is usually far less.

The necessity for reenforcing a concrete footing with steel bars or expanded metal, originated from the desire to secure a footing with a great spread or area and a minimum depth between the points *b* and *c*, Fig. 18. In meeting these conditions, the batter *dc* of the footing was necessarily made less than 60°, the safe slope for any masonry footing, and the footing in consequence became a slab or raft instead

of a pier. A concrete footing of the dimensions shown in Fig. 18, without the embedded steel rods a, a, would fail under a heavy load by transverse stress along the line fg, but by the introduction of metallic reenforcement,

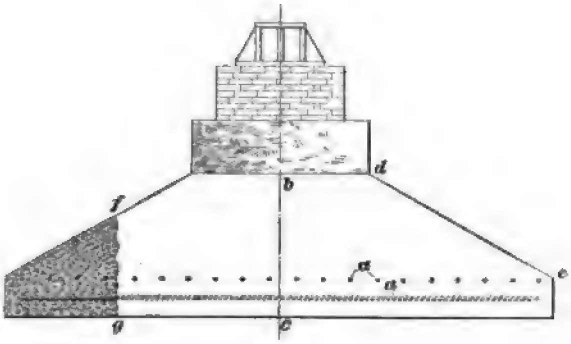
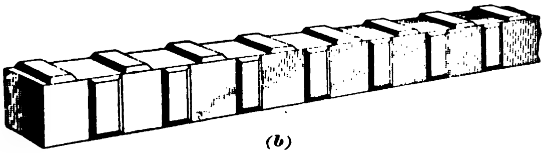


FIG. 18

the transverse strength of the footing is greatly increased and the required resistance is supplied.

For the reenforcement of concrete footings, it is customary to use square wrought-iron or steel bars, though in some instances several layers of expanded metal have been employed with success. When wrought-iron or steel bars are used, it is advisable to twist or indent them in

(a)



(b)

FIG. 19

order to increase the adhesion between the mortar and the concrete. When the twisted bars shown in Fig. 19 (a) are used for the reenforcement of concrete, it is known as the *Ransome system* of construction, and, as may be observed

from the figure, when the concrete has set around such bars, it requires a great force to destroy the adhesion between the two materials. The same purpose is accomplished, however, by corrugating the bars as shown in Fig. 19 (*b*). These bars are rolled with ribs or corrugations as illustrated, and have been patented by the Expanded Metal Company. They have been successfully employed in the construction of reenforced footings and fireproof-floor systems. The particular advantage claimed for the corrugated bar in the construction of reenforced concrete footings is that the sides of the projections on the bar make an angle with a plane at right angles to the axis of the bar that is less than the angle of friction between concrete and iron or steel.

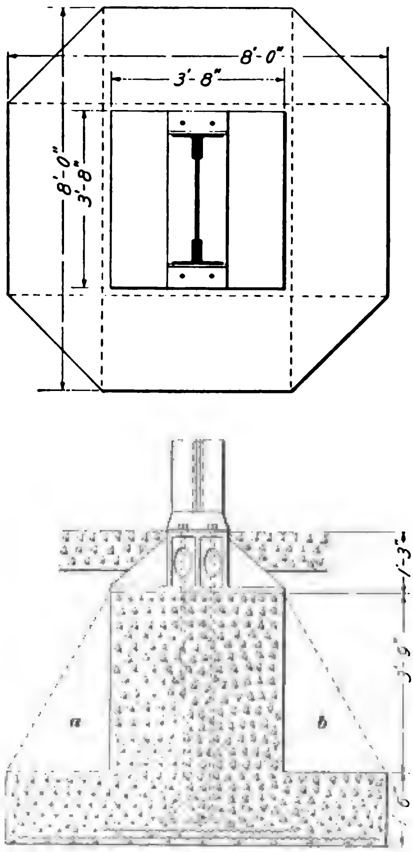


FIG. 20

18. A spread footing of concrete reenforced with expanded metal is shown in Fig. 20. In the design of this column foundation, two sheets of expanded metal are crossed as shown by the dotted lines in the plan and as further illustrated in the section. By the use of the expanded metal in this manner a considerable saving in concrete may usually be obtained, for the same footing, to comply with the

requirements of good practice, would be of solid concrete, filling in the spaces *a* and *b*.

19. In the design of reenforced concrete footings, it is evident that it is first necessary to decide on the area of the footing bearing on the soil and then to determine the depth of concrete required at the section of maximum bending moment, as well as obtaining, by calculation, the number of reenforcing bars necessary in order to realize the full strength of the concrete footing.*

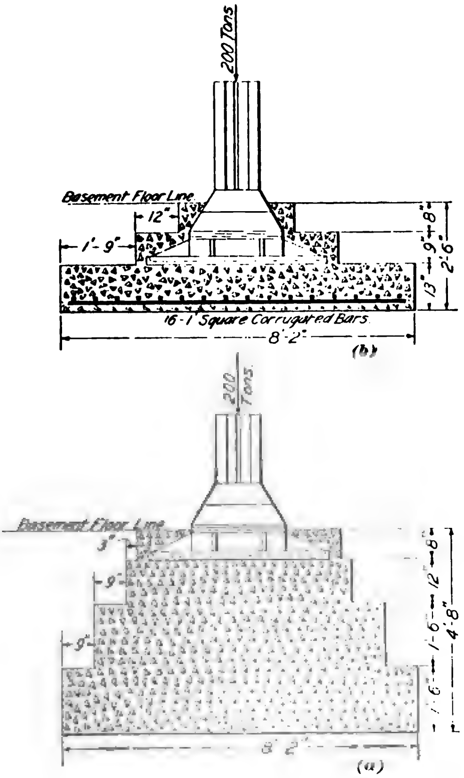


FIG. 21

The first factor is readily determined when the load of the column is known and the bearing value of the soil has been ascertained. The other two, however, must be determined by applying the principles governing the strength of materials and the method for determining the thickness of the concrete; the number of iron bars required at the section of maximum bending moment will be discussed in the following pages.

20. The possible economy introduced by the employment of a reenforced concrete footing instead of one of solid concrete is graphically shown in Fig. 21 (*a*) and (*b*). In (*a*) is shown a solid concrete footing having a batter that

approximately approaches an angle of 30° with the vertical. In view (*b*) is shown a reenforced concrete footing of the same area. Its height, however, from the basement-floor line to the bottom of the footing, is 2 feet 6 inches as compared with the height of 4 feet 8 inches for the construction shown in (*a*). These two footings were designed by the Expanded Metal Company, and the comparison of cost made as follows.

Estimate of cost for the solid concrete footing shown in Fig. 21 (*a*):

Excavation, 11.53 cubic yards @ 75 cents .	$ 8.65
Concrete, 202.3 cubic feet @ 26 cents . . .	52.60
Total cost of solid concrete footing . . .	$ 61.25

Estimate of cost for the reenforced concrete footing shown in Fig. 21 (*b*):

Excavation, 6.18 cubic yards @ 75 cents .	$ 4.64
Concrete, 86 cubic feet @ 26 cents	22.36
Extra length of column, 53 pounds @ 3.5 cents	2.91
Corrugated bars, 720 pounds @ 3 cents . .	21.60
Total cost of reenforced concrete footing	$ 51.51

Both of the footings shown in Fig. 21 have been proportioned to sustain a column load of 200 tons. A similar comparison of costs between reenforced concrete footings and grillage foundations composed of concrete and steel beams, shows that the corrugated-bar construction can be introduced for one-half to two-thirds the cost of the beam construction.

21. In Fig. 22 (*a*) and (*b*) are shown the designs for footings of equal dimensions and strength. In view (*a*) is shown a concrete footing reenforced with corrugated bars, each indicated by a single line. In the plan view, sections are taken on different planes to show the different layers of corrugated bars. In (*b*) is shown a footing subjected to the same conditions but built on the steel-beam grillage plan. The comparative estimate of costs of these two footings is given in the following bill of materials.

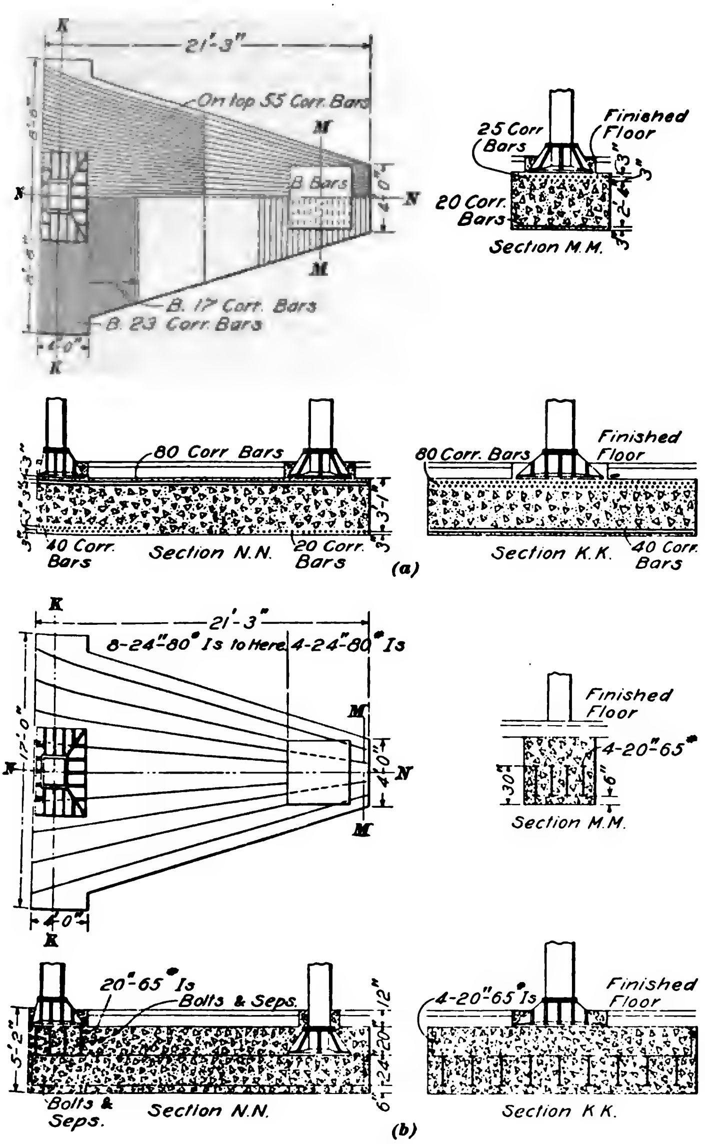

21'-3"
On top 55 Corr. Bars
B Bars
B. 17 Corr. Bars
B. 23 Corr. Bars
4'-0"
25 Corr. Bars
Finished Floor
20 Corr. Bars
Section M.M.
80 Corr. Bars
40 Corr. Bars
Section N.N.
20 Corr. Bars
(a)
80 Corr. Bars
Finished Floor
Section K.K.
40 Corr. Bars
21'-3"
8-24"-80# Is to Here
4-24"-80# Is
17'-0"
4'-0"
Finished Floor
4-20"-65#
Section M.M.
20"-65# Is
Bolts & Seps.
5'-2"
Bolts & Seps.
Section N.N.
(b)
4-20"-65# Is
Finished Floor
Section K.K.

Fig. 22

Estimate for the reenforced concrete footing shown in Fig. 22 (*a*):

Concrete, 793 cubic feet @ 26 cents . . .	$206.18
Corrugated bars, 5,919 pounds @ 3 cents .	177.57
Total cost	$383.75

Estimate for the steel-beam grillage footing shown in Fig. 22 (*b*):

Concrete, 985 cubic feet @ 26 cents . . .	$256.10
Steel beams, 16,760 pounds @ 3 cents . .	484.80
Bolts and separators, 800 pounds @ 3 cents	24.00
Total cost	$764.90

In this comparison of costs between the reenforced concrete footing and the steel-beam grillage, the excavation was not taken into account, as there is little difference in this item between the two constructions.

REENFORCED CONCRETE FOOTINGS FOR A SINGLE-COLUMN LOAD

22. Where a spread footing of the reenforced concrete type supporting a single-column load, Fig. 23 (*a*), is employed, the projecting portion of the concrete slab forming the base of the footing is subjected to transverse stress. It is a difficult matter, however, to decide the lines of fracture that the lower concrete slab will assume when subjected

that are so liable to failure, in the calculations for figuring the transverse strength of projecting footing courses beneath column foundations. In designing reenforced concrete footings, it is reasonable, when the above assumption is made, to consider that the slab of concrete forming the footing tends to fail along the line *a b*, Fig. 23 (*c*), and that the pressure of the soil, tending to break the concrete along this line, acts on an area equal to the stippled portion in the figure. On these premises the bending moment may be calculated and the thickness of the concrete and the reenforcing rods decided. The bending moment on the projecting footing course is equal to the moment of the pressure of the soil beneath, acting through a lever arm equal to the distance from the center of gravity of the trapezoid *a b c' d'* to the line *a b*. If the footing is square in plan, the pressure on the portion *a b c d* should be considered as the total pressure on the portion *a b c' d'*. Therefore, if *P* equals the total pressure on the trapezoidal projection and *l* equals the distance from the line of rupture *a b* to the center of gravity of the trapezoidal

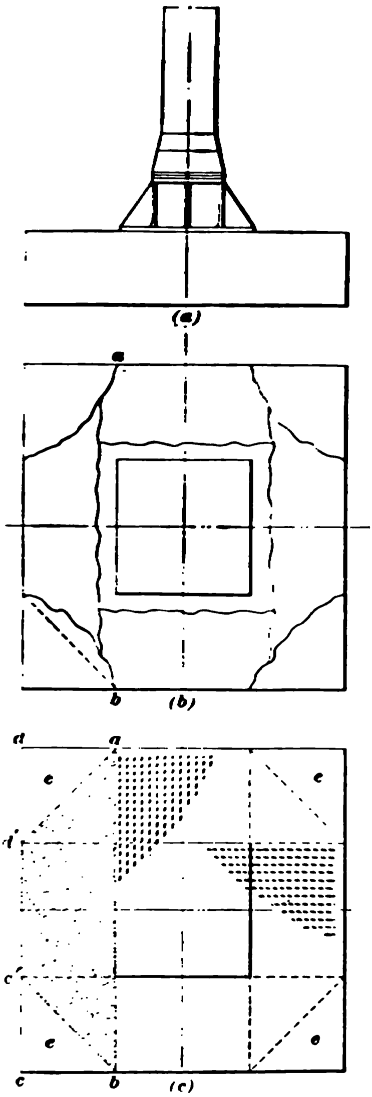


Fig. 23

area, the formula for the bending moment on the projecting portion will be $M = Pl$. As the resisting moment must equal the bending moment, M, may be represented by the same formula, or Pl.

The concrete should be of such thickness and the reenforcing bars should be so spaced that a projecting portion of the footing will have a factor of safety of at least 4.

The methods for determining the thickness of the concrete and the number of reenforcing bars required are explained under the following heading.

REENFORCED CONCRETE FOOTINGS SUPPORTING TWO COLUMNS

23. In proportioning **reenforced concrete footings supporting two columns**, it is imperative that a line representing the resultant pressure of the soil approaches or coincides with the resultant line of action of the superimposed loads. By this it is meant that, if the two loads were placed on a beam balanced at a single point, this point would be situated adjacent to a line that represents the line of action of the resultant pressure of the soil on the footing area.

In proportioning footings supporting two column loads, it is seldom possible to so design the footing that the line of action of the resultant of the two loads will exactly coincide with the line of action of the resultant pressure of the soil. Where this condition does fortunately exist, the footing exerts a uniform pressure on the soil throughout its area. Where, however, the two forces do not coincide, the pressure is not uniform throughout the footing area, but varies somewhat between the ends of the foundation. This variation of pressure should never be sufficient to cause unequal settlement, but it must be considered in the problem for determining the number of reenforcing rods that it is necessary to supply to the concrete in order to attain the requisite strength.

24. A concrete footing supporting two structural steel columns on cast iron bases is shown in Fig. 24, the elevation of the construction being above and the plan below. In the

design of this footing, the plan has been laid out as shown, in order that the pressure on the soil may be as nearly uniform as possible, for if the footing had been made rectangular in shape there would have been considerably greater pressure under the left-hand column than would have existed under the right-hand column, which is not so heavily loaded.

In order that the slight variation in the pressure beneath the footing may be determined, the relative positions of the

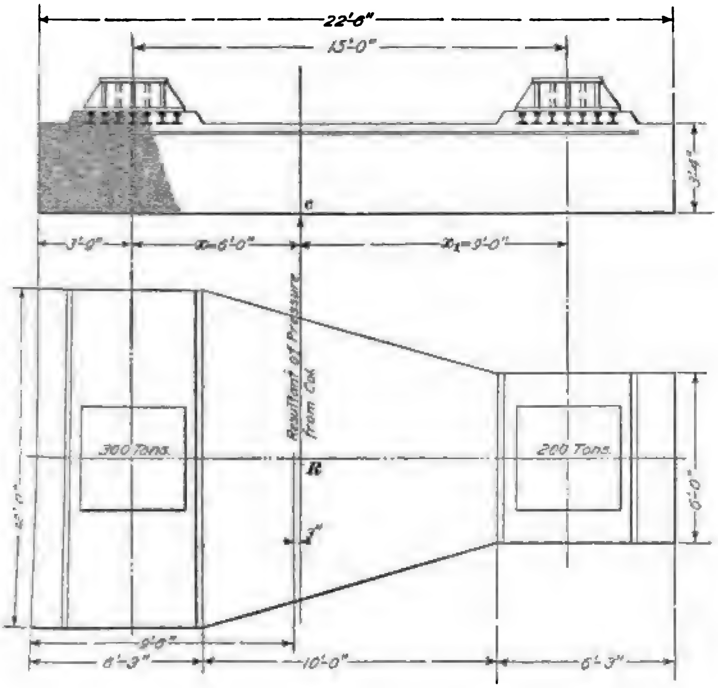


FIG. 24

resultant pressure from the loads and the resultant pressure from the soil must be obtained. If they coincide, as previously stated, the pressure on the foundation footing will be uniform, but if they are not located in the same position there will be a moment of the one resultant about the other; and when this moment has been found, the extremes of the variation in pressure on the footing may be

determined in a similar manner to that employed in finding the extreme fiber stress on a beam section.

In order to explain this further, the analysis will be applied to the footing shown in Fig. 24; in this case, the left-hand column supports a load of 300 tons while the right-hand column sustains 200 tons. These two pressures act in the same direction and may be balanced by a single opposing force, provided that it is located at such a position that rotary equilibrium will exist. The diagrammatical sketch shown in Fig. 25 represents the column loads as W_1 and W_2 and the resultant and opposing force as R. The position of

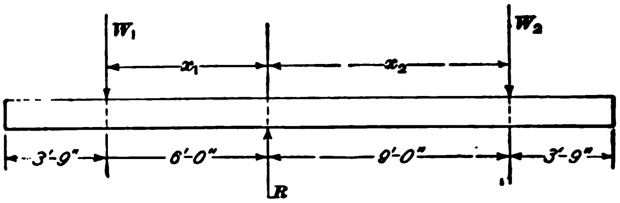


FIG. 25

R, which it is desired to determine, is the position of the resultant of the two column loads. From the principle of moments it is known that $W_1 x_1$ must equal $W_2 x_2$, or that $x_1 = \frac{W_2 x_2}{W_1}$; the same law is expressed by the proportion $W_1 : W_2 = x_2 : x_1$. This proportion is expressed by the statement that the distances from the two loads to the resultant R vary inversely as the ratio between the two loads.

In the example under consideration, the ratio between W_1 and W_2 is as 3 to 2, so that the entire distance between the column centers is represented by 5. In consequence, the distance $x_1 = \frac{2}{5}$ of 15 feet, or 6 feet, while the distance $x_2 = \frac{3}{5}$ of 15 feet, or 9 feet. A single force, therefore, located along a line at a distance of 9 feet 9 inches from the left hand end of the footing and equal in amount to the sum of the loads on the columns, will exactly balance these loads and the system of forces will be in equilibrium.

25. The factor now to be determined is the position of the center of pressure from the soil; this must be located

along a line through the center of gravity of the footing area. The position of this line may be found according to the principles employed in calculating the properties of sections. The formula by which this calculation may be made is expressed by the equation $c = \frac{\Sigma a d}{A}$, in which c equals the distance from the outside edge of the footing to a line parallel with this edge passing through the center of gravity of the area; a, the area of each elementary section; d, the distance from the center of gravity of the elementary section to the outside edge under consideration; and A, the total area of the footing plan. The sign Σ implies the sum of, and signifies that the products obtained by multiplying each elementary area by its distance from the edge are to be added. In order, therefore, to calculate the position of a line passing through the center of gravity of the footing area for the problem under consideration, the footing plan is redrawn as shown in Fig. 26 (a); in this figure each elementary section is represented by the different section lines. The first section to the left can be denominated by A, the central

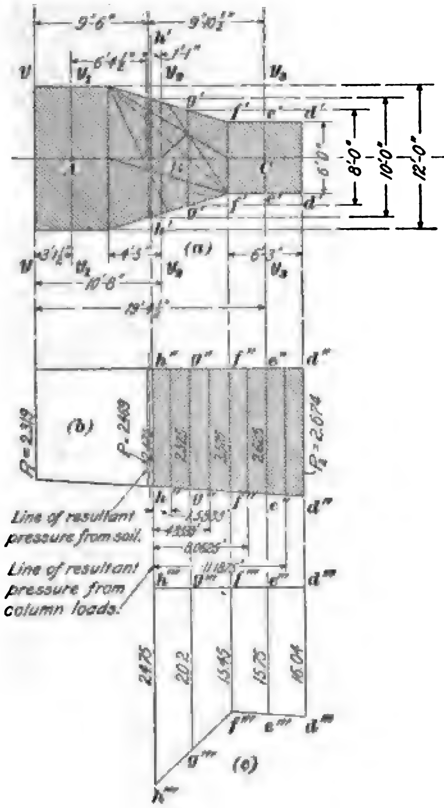


FIG. 26

section by B, while the right-hand section is designated as C. The center of gravity of the portion A is on the line $y_1 y_1$; the center of gravity of the portion B is on the line $y_2 y_2$; and the center of gravity of C is on the line $y_3 y_3$. The portions A and C are rectangles, so that their centers of gravity are located at the center of the areas, but the central portion B is a trapezoid and the distance of the center of gravity from one edge or the other must be determined either by calculation or by some graphical method. In this case a graphical method will be applied; this consists of dividing the trapezoid, by a single diagonal, into two triangles. When these triangles have been obtained as shown, the center of gravity of each triangle is readily found by drawing lines in each triangle from two angles to the centers of the opposite sides, the point of intersection being the center of gravity of the triangle. Connect these centers with a single line, shown heavy in the figure, and the intersection of this line with the horizontal center line of the figure will locate the center of gravity of the trapezoid on the line $y_2 y_2$, which is found to be 4 feet 5 inches from the left-hand edge of the elementary figure. The distance d of the portion A is 3 feet $1\frac{1}{2}$ inches; of the portion B, 10 feet 8 inches; while the same distance for the portion C is 19 feet $4\frac{1}{2}$ inches. The areas of the elementary sections A, B, and C are equal, respectively, to 75, 90, and 37.5 square feet, and the total area of the footing plan is equal to 202.5 square feet. These several values having been obtained, the calculation for the distance c is as follows:

Moment of portion A about the line yy equals 75×3.125	234.3750
Moment of portion B about the line yy equals 90×10.6667	960.0000
Moment of portion C about the line yy equals 37.5×19.375	726.5625
Sum of moments about line yy	1920.9375

Since the distance c is equal to the sum of the moments divided by the total area, the value of c will be 1,920.9375

$\div$ 202.5 = 9.486 feet, which is practically 9 feet 6 inches. This line may then be located on the plan, as shown in Fig. 26, at a distance of 9 feet 6 inches from the line $y y$. The distance between the center of gravity of the footing area and the resultant of the column loads is therefore 3 inches, as shown in Fig. 24.

26. The condition of the reenforced concrete footing with regard to rotary equilibrium is as shown in Fig. 24. Here the resultant from the soil tends to rotate the footing about the center c contained in the line of action of the resultant of the column loads. Consequently, in order that the footing may be in equilibrium, there is an upward tendency at the left-hand end of the footing and a corresponding downward tendency at the right-hand end, the one detracting from the mean average pressure of the soil and the other increasing it from the line passing through the center of gravity of the footing area toward the right-hand edge.

The variation at the outside edges may be found by the formula $M = \frac{P I'}{c}$, in which the value M equals the moment of the one resultant about the other; P, the pressure at the extreme edges of the footings; I', the moment of inertia of the entire section; and c, the respective distances from the line passing through the center of gravity of the footing plan to the edges in question. By transposition, the value of P equals $\frac{M c}{I'}$. In the problem under consideration the value of M is the resultant pressure equal to the sum of the column loads multiplied by the lever arm of 3 inches, or $M = 500 \times .25 = 125$ foot-tons. The value of I' must be calculated and is equal to $\Sigma a d^2 + I$; that is, the moment of inertia of the footing area is equal to the sum of the products of each elementary area of the footing plan by the square of the distance of its center of gravity from the center of pressure of the soil or line passing through the center of gravity of the footing area, plus the moment of inertia of the elementary section. The calculation for the value of I' in this

problem is given below and may be followed by referring to the diagrammatic plan, Fig. 26.

$$I' = \Sigma a d^2 + I \begin{cases} A = 75 \times 6.375^2 + 244.14 = 3\,2\,9\,2.1\,9 \\ B = 90 \times 1.1667^2 + 722.22 = 8\,4\,4.7\,3 \\ C = 37.5 \times 9.875^2 + 122.07 = \underline{3\,7\,7\,8.9\,1} \end{cases}$$

$$\text{Or } I' = 7\,9\,1\,5.8\,3$$

The distance c for the left-hand portion of the footing plan is 9 feet 6 inches, while the corresponding distance for the right-hand portion of the footing is equal to 13 feet.

Now that the several values of c, M, and I' have been obtained, the value of P, or the extreme variations in pressure on the soil at either end of the footing, may be obtained by substituting in the formula $P = \frac{Mc}{I'}$, as follows:

$$\text{Value of } P \text{, at left-hand edge} = \frac{125 \times 9.5}{7{,}915.83} = .15 \text{ ton}$$

$$\text{Value of } P \text{, at right-hand edge} = \frac{125 \times 13}{7{,}915.83} = .205 \text{ ton}$$

The average unit pressure on the foundation soil is 2.469 tons, and therefore the effective pressure at the outside edges of the footing represented by P_1 and P_2 may be found by adding or deducting the variations in pressure, as found above, from the average pressure. In this manner, $P_1 = 2.469 - .15 = 2.319$ tons and $P_2 = 2.469 + .205 = 2.674$ tons. The actual pressure on the foundation soil may then be represented as in Fig. 26 (b). In this diagram the length of the line P_1 is made, by scale, equal to the unit pressure at this point on the foundation soil, while the pressure at the right hand edge of the footing is represented by the line marked P_2. It will also be noticed that the pressure, as measured by the line P, is equal, by scale, to the average pressure on the soil.

27. Owing to the shape of the foundation plan, not only does the pressure vary for each unit of surface measurement, but the area for any portion of the footing is variable as well, so that before the actual upward pressure from the soil tending to rotate the footing about the line of resultant of

the column loads can be ascertained, a diagram, such as is shown in Fig. 26 (*c*), must be drawn. In this diagram the lengths of the ordinates $d''' d'''$, $e''' e'''$, $f''' f'''$, etc. are obtained from the diagrams (*a*) and (*b*) by obtaining the products of the ordinates $d' d'$ by $d'' d''$ and $e' e'$ by $e'' e''$, etc., as in the following calculation:

$d' d'$, etc.		$d'' d''$, etc.		$d''' d'''$, etc.
6	×	2.674	=	16.044
6	×	2.625	=	15.75
6	×	2.575	=	15.45
8	×	2.525	=	20.20
10	×	2.475	=	24.75

When all of the ordinates $d''' d'''$, $e''' e'''$, etc. have been found in this manner, the maximum bending moment on the concrete footing may be ascertained by taking the positive and negative moments about the line of the resultant of the column loads. If the column load is considered as creating a positive moment, the pressure from the soil beneath must, on the other hand, be taken as supplying a negative moment. Considered in this way, with the line of the resultant of the column loads as the center of moments, the several lever arms for the negative moments are as shown in Fig. 26 (*b*). The calculation is as follows: Positive moment of right-hand column equals $200 \times 9 = 1{,}800$ foot-tons. Negative moments of soil from beneath are found by considering each area as a trapezoid, then multiplying the area by the distance of its center of gravity from the resultant of the column load and adding the several products, as follows:

Area of portion $d''' d''' e''' e'''$ equals

$$\frac{16.044 + 15.75}{2} \times 3.125 \times 11.1875 = 555.774 \text{ foot-tons}$$

Area of portion $e''' e''' f''' f'''$ equals

$$\frac{15.75 + 15.45}{2} \times 3.125 \times 8.0625 = 393.047 \text{ foot-tons}$$

Area of portion $f''' f''' g''' g'''$ equals

$$\frac{15.45 + 20.20}{2} \times 3.25 \times 4.8333 = 279.999 \text{ foot-tons}$$

Area of portion $g'''g'''\ h'''\ h'''$ equals

$$\frac{20.20 + 24.75}{2} \times 3.1667 \times 1.5833 = 112.685 \text{ foot-tons}$$

The total negative moment then equals the sum of the above, or 1,341.505 foot-tons.

Since, from the preceding, the positive and negative moments are found to equal 1,800 and 1,341.505 foot-tons, respectively, their algebraic sum is equal to 1,800 – 1,341.505 = 458.495 foot-tons, 916,990 foot-pounds, or 11,003,880 inch-pounds. This amount is consequently the greatest bending moment on the reenforced slab of concrete. If a factor of safety of 4 is desired, the resisting moment of the section through the footing at the point of rupture will necessarily be equal to 11,003,880 × 4 = 44,015,520 inch-pounds.

28. Having obtained the required resisting moment, the problem resolves itself into the determination of the proper thickness for the concrete, under the condition of loading proposed in the problem. Rational formulas for determining the thickness of the concrete are as follows:

$$c_1 = \sqrt{\frac{M_r}{\frac{5\,b}{12}\left(\frac{f\,E_c}{E_s} + s_a\right)}} \qquad (18)$$

$$c_2 = \frac{2\,f\,E_c}{3\,s_a\,E_s}c_1 \qquad (19)$$

in which c_1 = distance from neutral axis of footing section to extreme edge of concrete on compressive side of slab;

c_2 = distance from neutral axis to center of gravity of metallic reenforcement;

M_r = resisting moment required, in inch-pounds;

E_c = modulus of elasticity of concrete;

E_s = modulus of elasticity of metallic reenforcing material;

f = elastic limit for metallic reenforcing material;

s_a = safe unit compressive strength of concrete;

b = width of section, in inches.

Reference to Fig. 27 will explain the values c_1 and c_2, as they are marked on the section. In the concrete footing, the reenforcing bars are to be placed in the upper surface, for at that position exists the tension. This is due to the fact that the conditions are the reverse of those existing where a simple beam is uniformly loaded. In the case under consideration, the pressure of the soil on the under side of the footing corresponds to the uniform load on the simple beam while the columns take the place of the upward reactions; thus, the position of the tensile and compressive stresses is reversed.

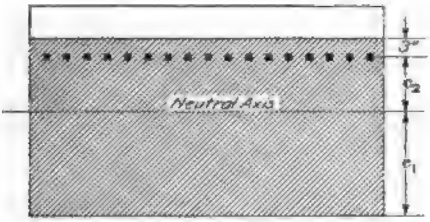


FIG. 27

Consequently, c_1 is the distance from the neutral axis to the under side of the footing and c_2 is the distance from the same line to the center of the reenforcing bars, to which distance must be added 3 or 4 inches, the necessary thickness of concrete to properly cover the metal. The distance from the neutral axis of the section to the top surface of the concrete slab forming the footing may thus be determined.

With reference to the problem, the value of M_1 has been found, by calculation, when the factor of safety is introduced, to equal 44,015,520 inch-pounds. The modulus of elasticity E_c of rock concrete in the proportion of 1, 2, and 6, though not accurately known, may safely be assumed at 4,800,000. The modulus of elasticity E_s for steel has been definitely ascertained and is usually taken at 30,000,000. The elastic limit f for medium steel, of which it is assumed the bars will be made, may be taken at 30,000 pounds, while the same value for the concrete, or s_a, is about 1,500 pounds. The width of the section of the footing along the line of rupture $h'h'$, Fig. 26, is found, by scale, to equal 120 inches, or more accurately, by calculation, 118.8 inches. The several values having been thus decided on, the values of c_1 and c_2 may

be found by substitution in formulas **18** and **19**, the calculation being made as follows:

$$c_1 = \frac{44{,}015{,}520}{\sqrt{\frac{5 \times 118.8}{12}\left(\frac{30{,}000 \times 4{,}800{,}000}{30{,}000{,}000} + 1{,}500\right)}} = 11.88 \text{ inches}$$

$$c_2 = \frac{2 \times 30{,}000 \times 4{,}800{,}000}{3 \times 1{,}500 \times 30{,}000{,}000} \times 11.88 = 25.344 \text{ inches}$$

When the distances c_1 and c_2 have been obtained, as in the above calculation, and 3 inches added in order to cover the reenforcing bars, the depth of the footing will be made up as follows:

Distance c_1	=	11.88 inches
Distance c_2	=	25.344 inches
Additional thickness	=	3.000 inches
Total thickness of footing	=	40.224 inches

29. As the thickness of the concrete footing has been obtained, all that remains to complete this problem is to determine the number of corrugated reenforcing bars required. In order to do this, their sectional area must be known as well as the distance the bars must be spaced from center to center at the section of greatest bending moment.

Table II gives the net section, in square inches, and the weight, in pounds per foot of length, of square corrugated bars.

TABLE II

NET SECTION AND WEIGHT OF SQUARE CORRUGATED BARS

Size Inches	Net Section Square Inches	Weight in Pounds per Foot
$\frac{1}{2}$	.19	.78
$\frac{3}{4}$	.38	1.56
$\frac{7}{8}$	.55	2.25
1	.70	2.90
$1\frac{1}{4}$	1.10	4.56

It is sometimes necessary to use the large sizes of bars, but generally the smaller sizes should be used, as by this means the metal is more evenly distributed throughout the concrete.

The distance between centers of bars may be represented by d and is found by the formula

$$d = \frac{1.6\, f\, a'}{s_a\, c_1} \qquad (20)$$

In this formula, d equals the distance that it is necessary

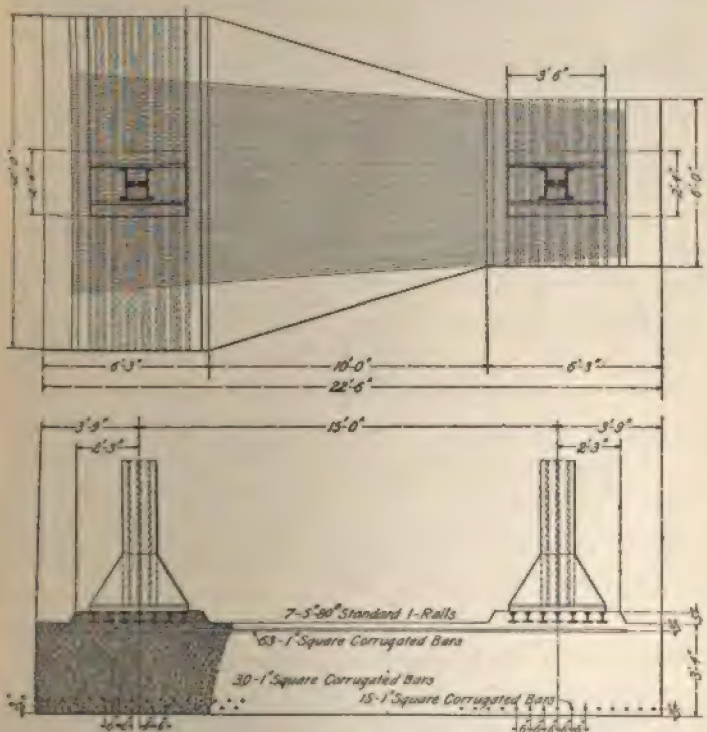


Fig. 28

to place the reenforcing bars apart in order to realize the conditions expressed in formulas **18** and **19**. The values of f, s_a, and c_1 are known, while the value of a',

which is the net sectional area for a 1-inch square corrugated bar, is .70 square inch, obtained from Table II. By substitution, then

$$d = \frac{1.6 \times 30{,}000 \times .70}{1{,}500 \times 11.88}$$
$$= 1.88 \text{ inches}$$

Since the width of the footing along the line of greatest bending moment equals 118.8 inches, the number of reenforcing rods will equal 118.8 ÷ 1.88 = 63.19.

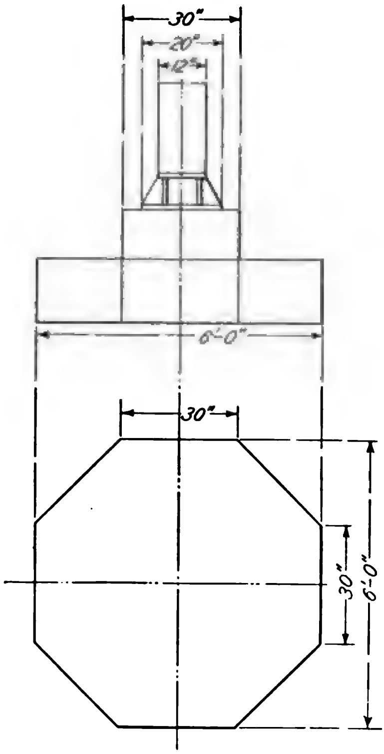


FIG. 29

All the data necessary for the design of the footing having been obtained in this manner, the working drawing may be laid out as shown in Fig. 28. In connection with the design of this footing it may be stated that the concrete may be reenforced transversely by placing rods across the footing under each column load as shown. In this figure a structural steel column base is shown while in Fig. 24 a cast-iron base is used.

EXAMPLES FOR PRACTICE

NOTE.—In these examples, the values for the elastic limit of the bars, the modulus of elasticity of concrete and of the reenforcing bars, and the unit compressive strength of the concrete may be taken the same as those used in the example worked out under Art. 28.

1. Determine the bending moment on the single column footing shown in Fig. 29, if a load of 200 tons is used. Ans. 902,010 in.-lb.

2. (*a*) Provided that the maximum bending moment is considered at the edge of the superimposed wall, how many steel corrugated

bars ½ inch square will be required per running foot to reenforce a concrete footing supporting a wall that carries 20 tons per foot? The wall is 30 inches thick and the concrete projects 18 inches on each side. (*b*) How many bars will be required if the point of greatest bending moment is taken at the center? (*c*) What will be the thickness of the concrete in each case?

Ans. (*a*) 3.49 bars; (*b*) 4.72 bars; (*c*) 8.53 and 10.49 in., respectively

3. Determine the maximum bending moment on the footing shown in Fig. 30.

Ans. 22,344,000 in.-lb.

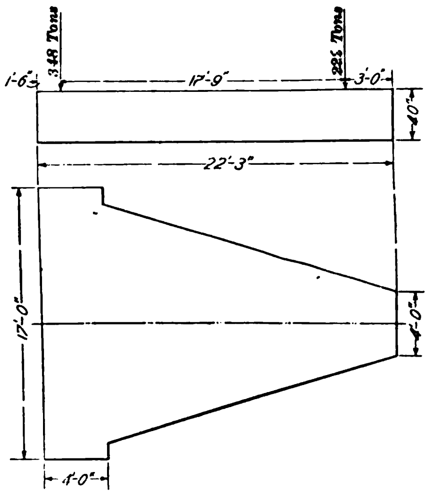


Fig. 30

4. Find the approximate number of 1-inch square corrugated bars required to reenforce the footing referred to in example 3; use a factor of safety of 3.

Ans. 87 bars

PILE FOUNDATIONS

30. Where the superstructure is of great weight and the soil of comparatively low bearing value, it is necessary to adopt some means by which great settlement will be prevented; to accomplish this, **pile foundations** are employed. These are constructed by driving piles at close intervals throughout the area of the footing and placing on them a capping of granite, concrete, or oak grillage.

The term *pile*, as usually accepted, is given to a long stick of timber, in fact, a portion of a tree trunk that is forced into the ground to form the foundation for a building, wharf, trestling, etc. However, piles may be of concrete, cast iron, or steel. When of cast iron they are cylindrical in form and taper somewhat, being reenforced on the end by thickening the metal at this point. Piles of steel usually consist of rolled shapes, such as I beams. Their use is limited,

however, for their cost is prohibitory when compared with timber piles, and they are seldom used for foundation work under buildings.

31. Piles may be classified according to the manner in which they are sunk or driven.

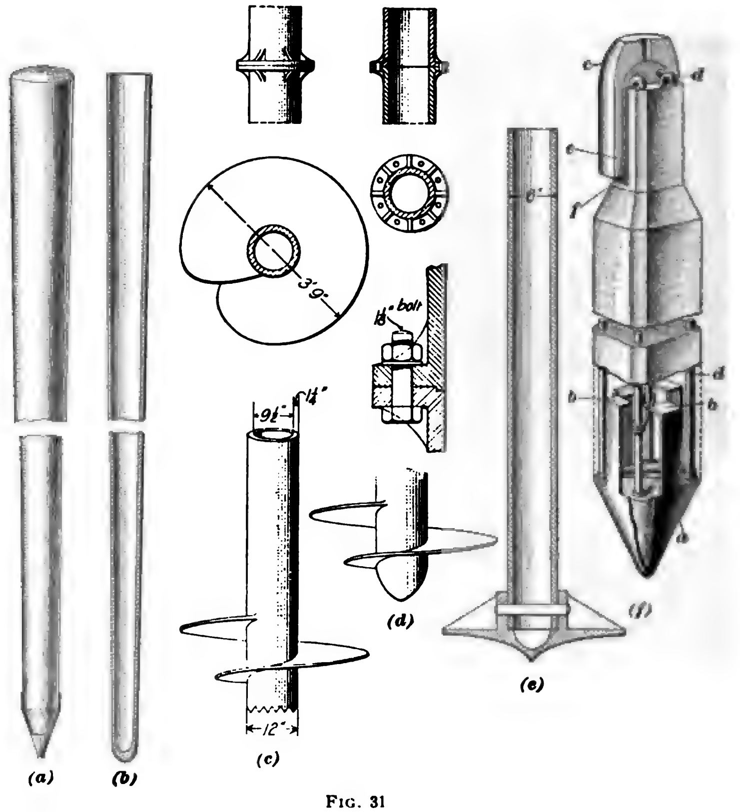


Fig. 31

The common *timber piles* for foundation work, as shown in Fig. 31 (*a*), also concrete, cast-iron, and steel piles, are in general driven by means of a hammer consisting of a weight dropping between guides from a height on the top of the pile. Timber piles should be closely trimmed, stripped of their bark, and sharpened at one end to facilitate driving. A

cast-iron pile is shown in Fig. 31 (*b*), but owing to the expense and difficulty in driving, these piles are seldom used.

Screw piles, as shown in (*c*) and (*d*), are those that are forced into the soil by means of a cast-iron screw at their lower end. This screw is usually of one or two turns and is securely keyed or setscrewed to the pile. Its projection from the pile may vary from 9 to 18 inches and its pitch should be from a quarter to a half of the projection. For most requirements, a screw having a diameter of from 3 feet 6 inches to 4 feet 6 inches will be found sufficient, a sandy foundation requiring the larger. The shaft of the pile may be of wood or steel, usually of the latter, in which case it consists of an extra heavy pipe from 6 to 12 inches in diameter. The lower end of the pipe is usually left open, the edge being beveled or provided with teeth to assist in cutting and penetrating the soil. Such piles may be used in most ordinary soils, for they will push their way through sand or gravel even when it contains boulders of some size. Hollow cast-iron screw piles are sometimes used and are frequently filled with concrete to strengthen them and increase the bearing surface. Screw piles are driven by a capstan and levers fixed on the top, and as the pile descends lengths of pipes are added.

The peculiar advantage of the screw pile lies in the fact that it can be used in sinking foundations adjacent to heavy footings that might be damaged by driving common timber piles with a hammer. It is sunk very rapidly through common soil and offers considerable resistance to both pressure and pulling.

Pneumatic piles are cylinders or caissons of steel or cast iron that are sunk by excavating from the inside. The pressure of air maintained within them prevents an influx of water. When the piles have been sunk to the proper depth and the interior excavated they are filled with concrete.

The *disk pile*, Fig. 31 (*e*), usually consists of a steel shaft or cylinder having keyed to the lower end a disk from 18 inches to 30 inches in diameter. These piles are sunk by washing away the earth from beneath the disk by a jet of

water under pressure. They are used for foundations in sand and for the substructure of piers and wharves.

The usual *concrete pile* is shown in view (*f*); it is made in molds at least 30 days before use, the concrete being reenforced by steel rods *d* connected with stirrups at short intervals.

Sheet piling consists of heavy planks or squared timber driven close together. This system of piling is employed to retain the earth or soil and prevent its spreading under the weight of the earth or adjacent footings. Its greatest use is to prevent the caving of earth into trenches caused by excavating for foundation footings or to prevent their inundation by subsoil or surface water. These sheet piles are usually provided with a tongue on one side and a groove on the other, so that each pile makes a tight joint with the adjacent ones.

32. The several types of sheet piling employed in building construction are shown in Fig. 32. A common form is composed of 3-inch or 4-inch tongued-and-grooved planks driven tight; this piling should only be used for light work, such as the lining of trenches for concrete foundations or for excavating in order to lay the footings of heavy piers or to provide elevator pits. The planks are sometimes put together with a **bird's-mouth** joint, as shown at (*a*), and as a stronger piling is thus formed this method should be used where practicable.

A patented type of sheet piling is shown in view (*b*). This consists of three thicknesses of rough timber planks bolted together and so arranged that a tongue and groove is formed by the placing of the center plank beyond the two side planks. This makes an excellent sheet piling, and when carefully driven usually will practically preclude the percolation of water. Where very heavy sheathing is required, square timbers are driven close together. Tightness is procured, if desired, by covering the heavy timbers with ceiling or by calking between with oakum, pitch, or red lead.

Sometimes sheet piling is made sufficiently tight to resist

the percolation of water by placing back of the sheet piling a 12-inch or 18-inch puddle wall of clay well tamped in place. In driving sheet piling, the planks and timbers should be driven between guides that are bolted to a frame properly supported and secured to posts or timber piles at frequent intervals, as shown in Fig. 33. Sheet piling is usually cut at the ends, as shown in Fig. 32 (*b*), so that the pile will be

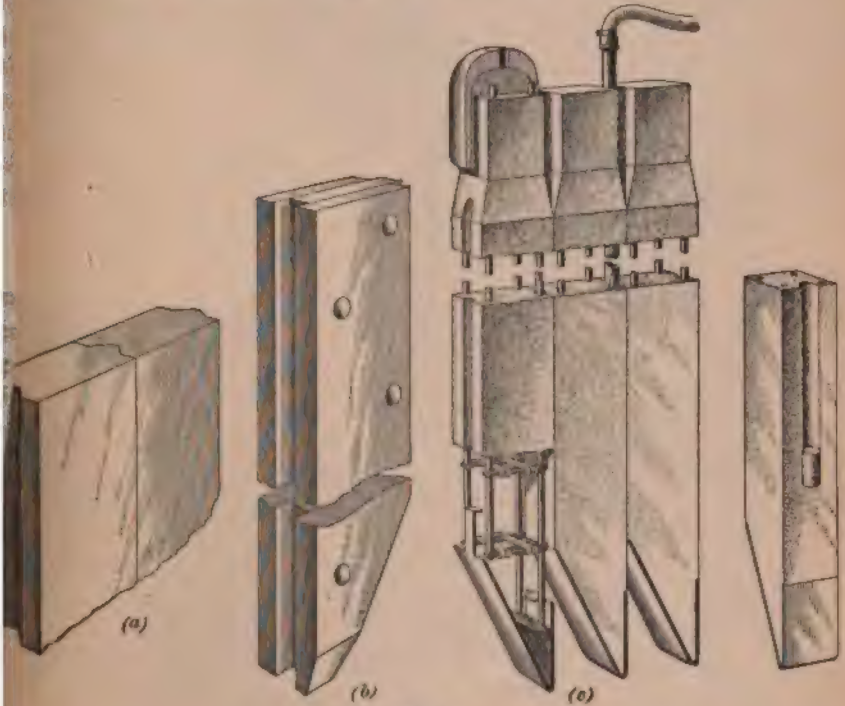


FIG. 32

forced against the adjacent piling and likewise against the guide frame, the pile always tending to force itself in a direction away from the bevel cut. The reenforced concrete sheet piles shown in view (*c*) are strengthened by means of four rods connected with wire clamps, the latter being cross-tied by flat irons. A projection is left near the base of each pile and a groove runs on both sides from this projection to

the top, so that, in driving, the projection on one pile slides in the groove of the last one driven.

An iron pipe that fits the grooves of two adjacent piles is connected by means of a hose with a pump, or water tank. This pipe serves as a guide, and the sand which might jam

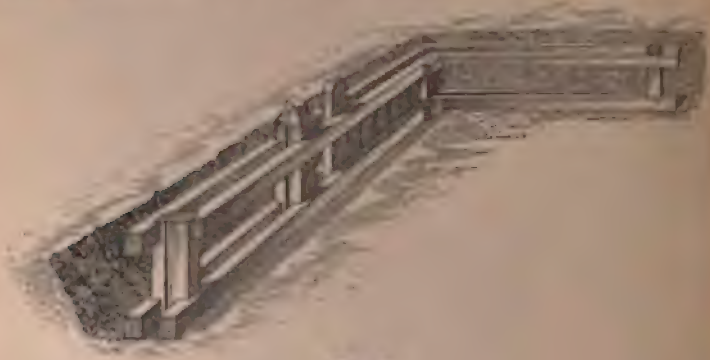


FIG. 33

the grooves is forced out by the water. After the pile is driven, this pipe is withdrawn and a water-tight joint secured by filling the grooves with cement.

Concrete sheet piles are recommended for marine work, such as wharves, docks, etc. Owing to the elasticity of the material the damage from blows of vessels is less than on wooden piles.

The common timber pile particularly interests the architectural engineer, and though at times he may resort to the screw pile or one of the special piles mentioned, it is seldom he can employ them. He should, however, have a general knowledge of them all in order to cope with exigencies that may arise.

TIMBER PILES

33. Timber piles are most durable when they are driven to such a depth that they will always be immersed in water; for this reason their tops are cut below low-water mark. In ordinary soils that admit of easy driving, spruce or hemlock piles are used for foundation work. For more

compact and harder soils hard pine, elm, or birch are used. Where the piles are not always immersed and are subjected alternately to wet and dry conditions, yellow pine or post oak is employed.

Wooden piles for building foundations should be spaced not more than 36 inches nor less than 20 inches from center to center, and they should be of such a size that the least dimension at the small end is 5 inches and the greatest dimension at the large, or butt end, is 12 inches. Where piles are over 20 feet long the butt end should be at least 20 inches in diameter.


(a) (b) (c)
FIG. 34

Before driving the pile, the small end should be beveled off to a blunt point; and if there is a liability of its splitting it should be provided with a wrought-iron strap, as shown in Fig. 34 (*a*), or with a cast-iron shoe similar to either of those shown in (*b*) and (*c*).

The top of the pile may be prevented from brooming, to a certain extent, by placing a wrought-iron ring about 1 inch thick and 3 inches wide on the end and driving it on by one or two light taps from the hammer. The ring should fit tightly so that it will not be readily displaced.

CONCRETE PILES

34. The advantage that **concrete piles** possess over timber piles is that the former are not affected by the rise and fall of water nor by the attacks of teredoes.

The lower end has a pointed shoe *a*, Fig. 31 (*f*), the side plates of which are turned in, as at *b*, *b*, for further security. While driving, the head is projected by a steel cap *c* previously filled with sand, thus forming a cushion that distributes the

pressure of the blow from the hammer. The head should be of less diameter than the body of the pile to allow a clearance for the application of the steel cap. By this arrangement, the iron rods *d* may be allowed to project above the head so that they may be connected with other parts of the structure. The cap is closed at the lower end by a clay ring *e* held by a plug of hemp or spun yarn *f*.

BEARING RESISTANCE

35. Piles offer the most secure foundation and one that has no settlement when they are driven to bed rock. The accomplishment of this is not always possible on account of the great distance below the surface at which the bed rock is found in some localities. When the bed rock cannot be reached with piles of a length obtainable, they are driven to hard pan, and in some cases are merely driven 20 or 30 feet into the clay.

Particular care should be taken to ascertain the nature of the foundation soil before driving the piles, and test borings should be made to determine the distance to bed rock or hard pan and the depth and nature of the several strata that may exist at the site. In some instances, piles have been driven with great difficulty through the upper stratum, repeated blows driving them only a fractional part of an inch, showing that they had great bearing resistance. Continuous driving, however, caused them to penetrate the upper stratum into a layer of soft underlying clay or quicksand, when, with each blow of the hammer, the pile was driven 10 or 11 inches and its bearing strength thus greatly impaired.

36. The **bearing value** of piles driven to bed rock is their ability to resist compression as a column. Piles driven to bed rock through a very compact soil are not likely to fail by lateral deflection but will fail by crushing. The pile may then be proportioned to the allowable resistance to crushing at the minimum section, and a factor of safety of at least 6

should be used. The building laws in several of the largest cities, however, stipulate that no pile shall be loaded in excess of 40,000 pounds.

Piles when not driven to hard pan or rock depend for their bearing value on the bearing value of their ends, and principally on the frictional resistance between their surfaces and the soil. The bearing value for piles not driven to bed rock is difficult to determine. Many rules and formulas have been evolved and the best are founded on practical data obtained by experiment, but they vary greatly and the engineer must choose the one that seems most nearly to agree with his experience. The following formula, which is conservative, is used in the best practice and is recommended by the building laws in several cities.

$$W_p = \frac{2\,WH}{p+1} \qquad (21)$$

in which W_p = safe load in tons, or bearing value of pile;
W = weight of hammer, in tons;
H = fall of hammer, in feet;
p = penetration of pile, in inches, produced by last blow of hammer.

The penetration of the pile under the last blow is considerably affected if the head is broomed or splintered in driving, and in order to obtain correct results when driving test piles these broomed heads should be removed before the last blow is struck.

37. Table III gives a statement of several of the principal formulas, though the one given above agrees more nearly than most of them with the actual tests made on piles, except possibly the one evolved from experiments by Hertiz.

In the formulas in Table III,
W_p = safe bearing value of pile, in tons;
W = ultimate bearing value of pile, in tons;
P = weight of hammer, in tons;
H = drop of hammer, in feet;
p = penetration of pile, in feet, under last blow.

EXAMPLE.—In driving the piles for the foundation of a large building, the penetration is ½ inch under the blow of a 1-ton hammer. What will be the allowable bearing value of the pile, figured according to formula **21**, provided the hammer falls 15 feet?

SOLUTION.—In formula **21**, the safe bearing value of the pile is $W_p = \frac{2WH}{p+1}$, and, by substitution, $W_p = \frac{2 \times 1 \times 15}{.5 + 1} = 20$ tons. Ans.

TABLE III

EMPIRICAL FORMULAS FOR DETERMINING THE BEARING VALUE OF TIMBER PILES DRIVEN BY HAMMER

Authority	Formulas
Trautwine . . .	$W = \frac{52 P^2 \sqrt{H}}{1 + 12p}$
Baker	$W = 100\left(\sqrt{PH + 50(p)^2} - 50p\right)$
Sander	$W_p = \frac{PH}{8p}$
Hertiz	$W = \sqrt{500\,PH + (250p)^2} - 250p$

EXAMPLES FOR PRACTICE

1. What will be the safe bearing value of a timber pile that under the last blow of a 1½-ton hammer has a penetration of 1 inch, the fall of the hammer being 12 feet? Ans. 18 T.

2. A pile driven through stiff clay has a penetration of 2 inches under the last blow of a hammer weighing 1,000 pounds and falling through a distance of 14 feet. What will be the allowable bearing value of the pile? Ans. 4.6 T.

---

38. In the construction of timber pile foundations it is first necessary to calculate the weight on the footing. Then, by driving a test pile and determining the values required in formula **21**, the allowable bearing strength is ascertained and the number of piles required in a particular area is readily determined. If they are driven too close together they tend to force each other out of place and also destroy the cohesiveness of the soil and, consequently, its bearing value. Piles should never be driven closer together than 20 inches or farther apart than 36 inches on centers, according

to the New York building laws. There is no reason, however, why they should not be spaced farther apart than 3 feet, provided that a sufficient number can be driven at a greater distance, and the grillage or capping is designed to span the space from pile to pile. After the piles have been driven, they are usually sawed to a level and are capped with either a grillage of timber or large granite footing stones, or they are filled in between and over the top with concrete.

39. The several methods of constructing the capping are shown in Fig. 35 (*a*), (*b*), (*c*), and (*d*). In (*a*) is shown the usual grillage capping, which should be of hardwood laid below low-water level and composed of timbers not less than 6 inches thick. The courses of timber should be *drift-bolted* to the tops of the piles and securely tied to each other by notching or by wrought-iron straps, dogs, or drift bolts. On the top of the grillage a flooring of heavy planks is placed and the masonry laid on this. Drift bolting consists in driving a round bar of iron through holes previously bored in the timber, the holes having an area somewhat less than the section of the bar. However, they must not be so small that the timbers will be split in drifting. The usual drift bolt consists of a 1-inch square bar driven into a $\frac{7}{8}$-inch hole. The timber in a grillage must be laid close and have sufficient transverse strength to sustain between the piles the several courses of masonry in the footings.

When the foundation stratum is boggy, the form of timbering shown in view (*b*) is adopted in order to transfer the load to solid strata beneath. Stout piles of sufficient length to reach the hard soil are driven along the site and connected at the top by heavy timbers notched and spiked to them; these are then connected by similar timbers running crosswise, which are also notched and spiked to those below. The whole is then embedded in concrete as shown. In view (*c*) is shown the usual granite capping laid on the top of the pile. The granite blocks should be dressed on the bottom, though sometimes they are *spot-faced;* that is, finished on the small area that bears on the top of the pile. The use of

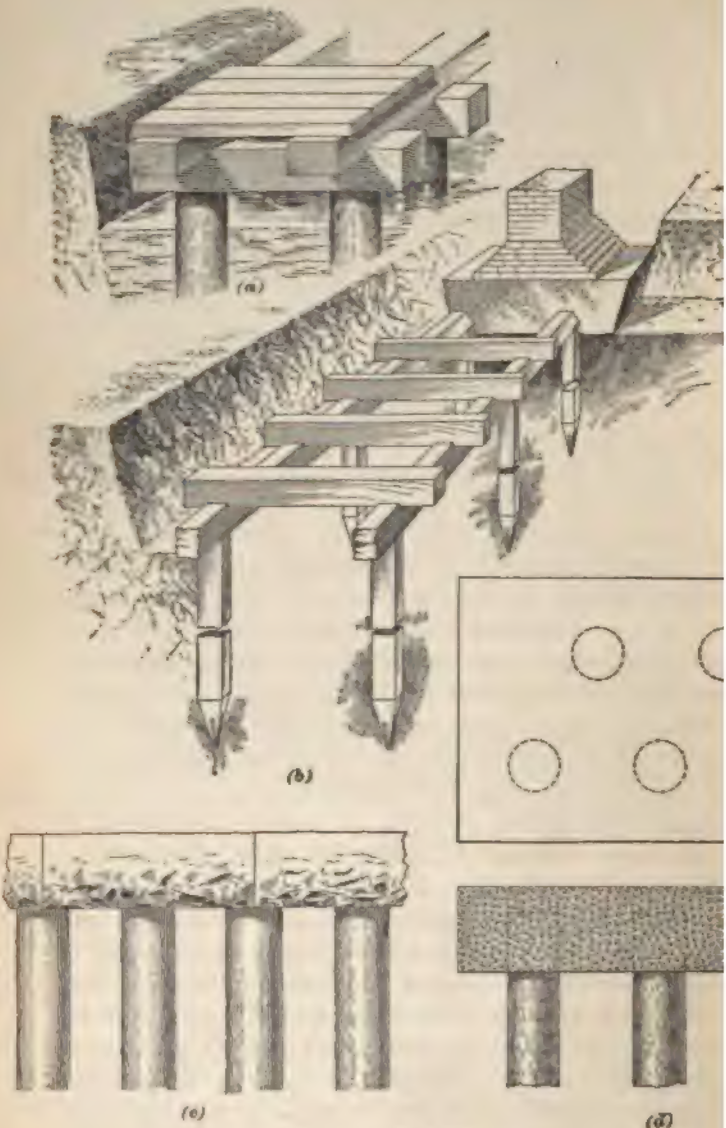


FIG. 35

granite capping makes possible a more durable footing than the use of wooden beam grillage, and it must be employed in cases where the beam grillage would be above the low-water line.

The most satisfactory capping for piles is concrete, as shown in view (*d*); when this is used it should be placed around the tops of the piles to a depth of at least 12 inches and should cover the sides of the piles to the same thickness; a layer of concrete 6 inches thick should be placed on top of the 12-inch layer to further reenforce it. Concrete has been adopted by many engineers in preference to timber grillage on account of its durability and because of the diminished possibility of the footings slipping from the timber platform by unequal settlement. They contend that this cannot occur with the concrete capping on account of its roughness and the adhesive surface that it offers for the bed of cement or mortar.

40. Where piles are driven through a yielding soil to hard pan or rock they are likely to be pushed over at the tops, moving about the lower end as a center. This can hardly occur with piles capped by concrete, but with a timber grillage or platform the precaution of filling in around the top of the pile with broken stone is often observed. Where this tendency toward lateral yielding is evident, a wall of piles should be driven around the foundation and the piles carrying the foundation secured to them by timber braces. Another method to prevent piles from moving laterally, though one that is hardly to be recommended, consists of tension braces or guy lines secured to plates or stones buried in the earth at some distance from the foundation. These guy lines or rods may be secured either to the piles or to the foundation wall; when to the latter, they are usually connected with an anchor that is built into the wall.

CANTILEVER FOUNDATIONS

41. In crowded sections of large cities, the building sites have great value, so that it is necessary, in order to obtain adequate remuneration on the investment, to utilize every foot of building area. It is sometimes possible to get good rental from basements and subbasements, so that it is quite common for an important building to have three or four floors below the ground level, these basement floors being supplied with artificial light and ventilation. Such unusual conditions require special features of construction; it is not uncommon in the erection of high buildings to employ what is known as **cantilever foundations.**

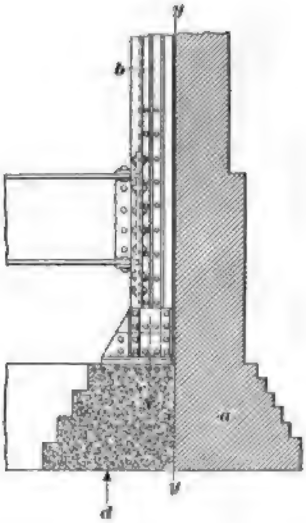


FIG. 36

42. In Fig. 36, *a* is the foundation of an old building that has a building of skeleton construction adjacent to it, close to the building line. It is evident that to build close against the building line *yy* and to get adequate foundation area for the support of the column *b*, all of the batter of the footing must be inside the building. In constructing the footings in this manner, the center of downward pressure from the column does not coincide with the center of upward pressure from the soil, and the condition shown by the arrows *c* and *d* exists. Therefore, it is evident that some special means must be provided to centralize over the foundations the concentrated loads from the outside columns, and at the same time utilize every inch

of ground surface by building close to the line. This condition is met by adopting the construction shown in Fig. 37. In this case the foundation adjacent to the wall column is carried on a concrete footing course *a*, and the entire weight of the outside wall of the new building adjacent to the existing wall at *b* is carried by the overhang of the girder at *c*. By this means the new wall is carried close to the old wall and the center of weight from above and the center of pressure from beneath on the footing coincide, as designated by the arrows. It is evident by inspection of Fig. 37 that the girder *d* is subjected to great bending stress

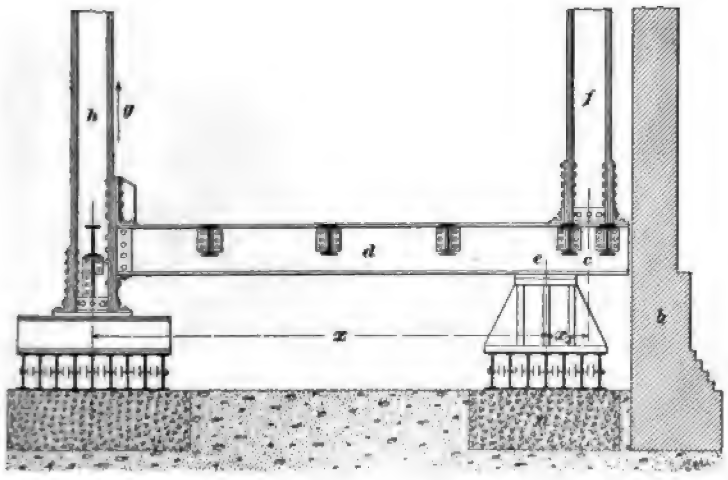


FIG. 37

at or adjacent to the point *e*, and that also if the wall and the loads supported by the outside column *f* are great there will be an upward lifting tendency as shown by the arrow *g*, so that the conditions of loading must be investigated to determine whether the weight on the column *h* multiplied by the lever arm x is equal to the weight on the column *f* multiplied by the lever arm x_1. Frequently, the overhang is so great and the dead loads on the interior column *h* are so light that it is necessary to securely anchor the column *h* to a heavy foundation that supplies the necessary weight to

prevent any tendency of the load on the overhang lifting the interior column.

43. Another condition that sometimes requires the adoption of the cantilever foundation is shown in Fig. 38. In order to obtain the headroom for the subbasement, it is necessary to excavate below the foundation of an old building adjacent to it. Such excavation would require either that the old building be underpinned and a new foundation carried down, as designated by the dotted lines, or that the foundation

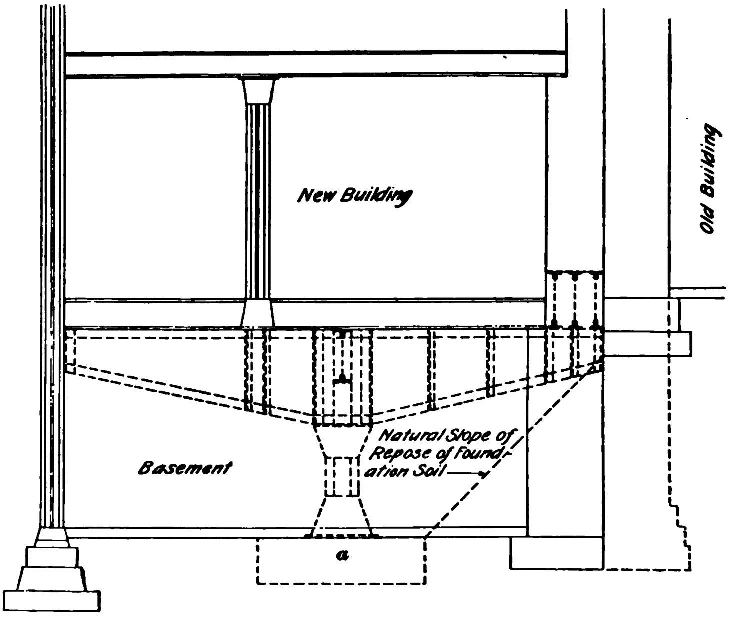


FIG. 38

footings for the new building be located further within the site. In order to do this, cantilever construction could be adopted, as shown by the dotted lines. When the construction designated is employed, the excavation for the footings *a* can be made without disturbing the footings beneath the old building, though such footings must not encroach on the natural slope of the soil on which the old footings are built, and which are shown in the figure by the dotted lines.

44. In Fig. 39 is designated a type of cantilever foundation used in a large office building. This foundation construction was used in order to bring the column loads over the center of gravity of the footings and to prevent the footings from overrunning the property line. Columns *abcd* are carried on cantilever girders *e*, *e*, which in turn are supported by a second cantilever girder *f*, this girder in turn being supported on distributing girders resting on a raft or grillage footing *g*. In this way the raft or grillage footing is practically symmetrically loaded and will produce a uniform bearing stress on the soil.

The several details of the cantilever grillage footing and column connection at the base are shown in Fig. 40. Over and under each set of girders where there is a concentrated load, the girder is well stiffened by vertical stiffeners *a a*, *b b*, and *c c*. These stiffeners are ground to fit between the flange angles of the girders. The girders *d d* are retained against any tilting tendency that might exist by the gusset-plate brace *e*. The space beneath, as constructed, is used as basement area for various purposes.

45. Another type of cantilever foundation construction is shown in Fig. 41. In this case, three-column footings are supported on a grillage, the two outside columns forming the support for a cantilever box girder that sustains the outside wall of the building. The plan of the foundation is shown in view (*a*) and the elevation in (*b*). The outside distributing girder sustaining the column that supports the cantilever is of heavy box-girder construction, while the distributing girders under the interior columns are 20-inch 85-pound I beams. The detail of the cantilever girder is shown in Fig. 42, and on examination is sufficiently clear as to details of construction without further explanation.

46. All of the details for the design of grillage footings beneath a cantilever foundation are given in this Section in Arts. **4** to **16**, and if a reenforced concrete footing is to be used in connection with the cantilever construction it may be proportioned by the data given in Arts. **17** to **29**. In

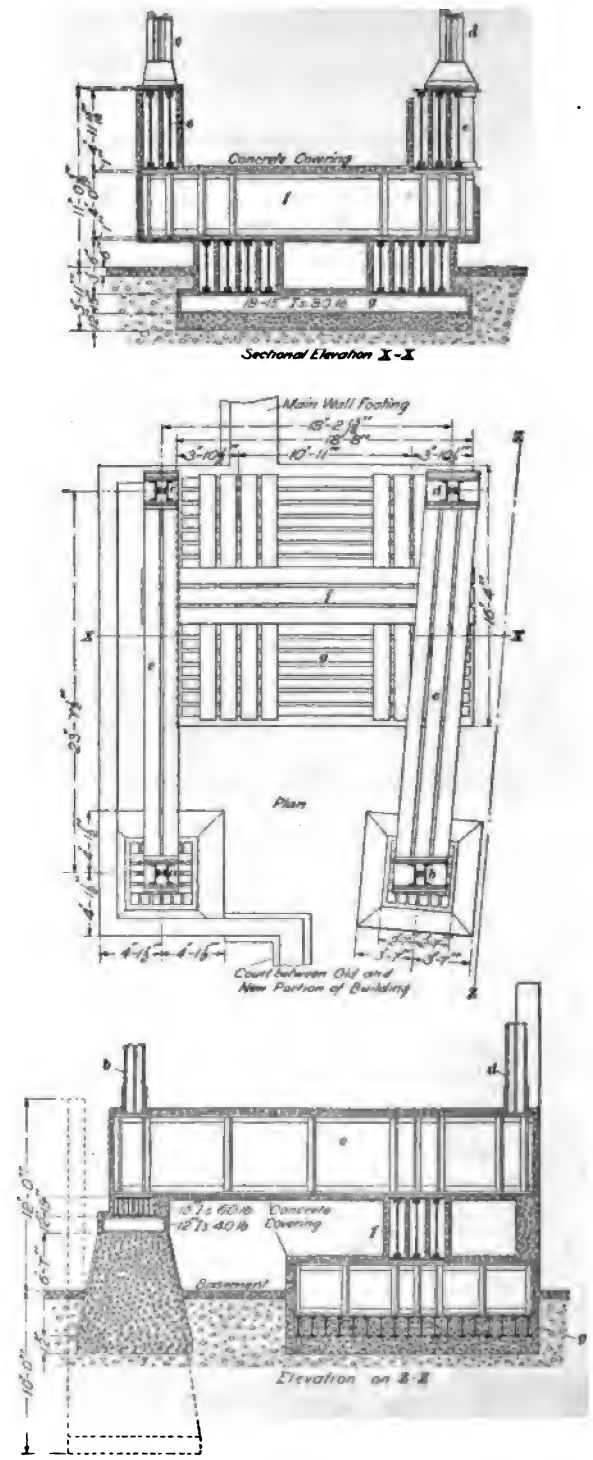

Concrete Covering
Sectional Elevation X-X
Main Wall Footing
Plan
Court between Old and New Portion of Building
Concrete Covering
Basement
Elevation on Z-Z

FIG. 39

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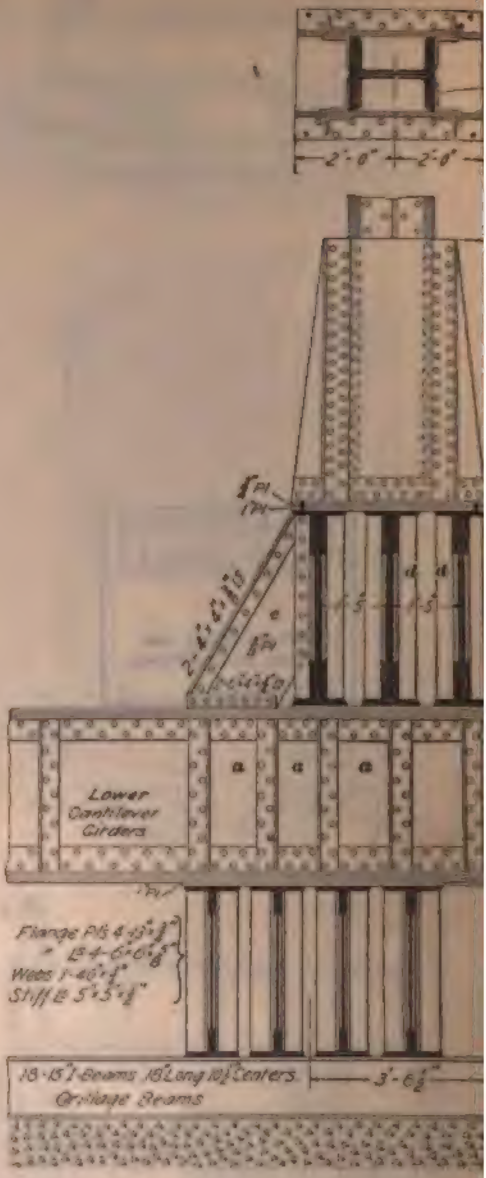
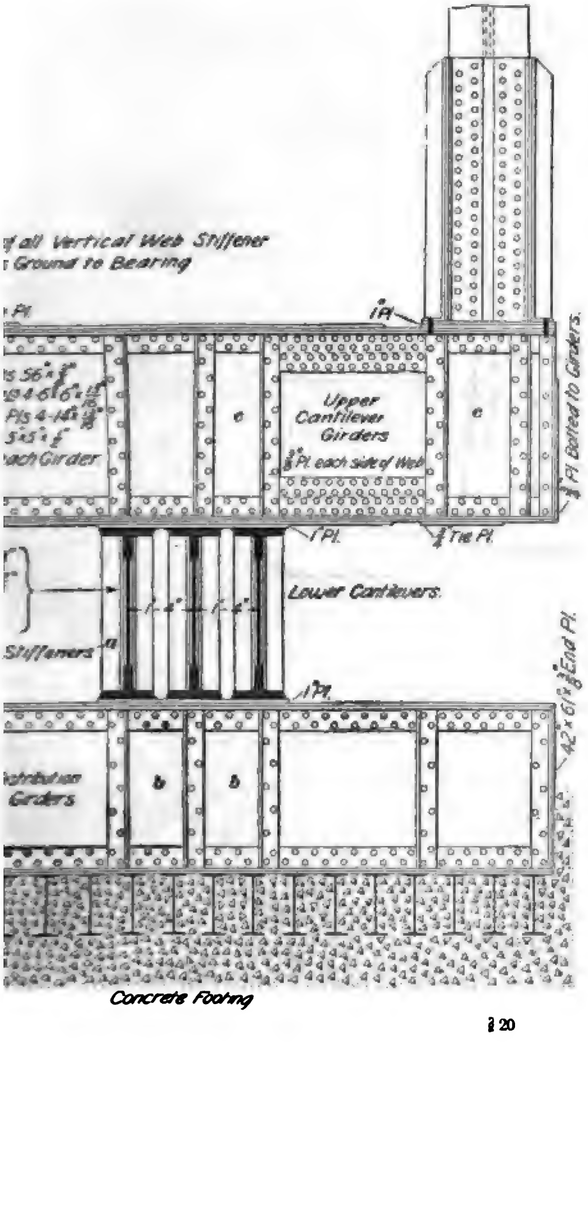
ASTOR, LENOX AND
TILDEN FOUNDATIONS.



2'-0"
2'-0"
Lower
Cantilever
Girders
Grillage Beams
3'-6½"



all Vertical Web Stiffener
Ground to Bearing
Pl.
Pl
Upper
Cantilever
Girders
Pl. each side of Web
Pl. Bolted to Girders.
Pl.
Tie Pl.
Lower Cantilevers.
Stiffeners
Pl.
End Pl.
Girders
Concrete Footing

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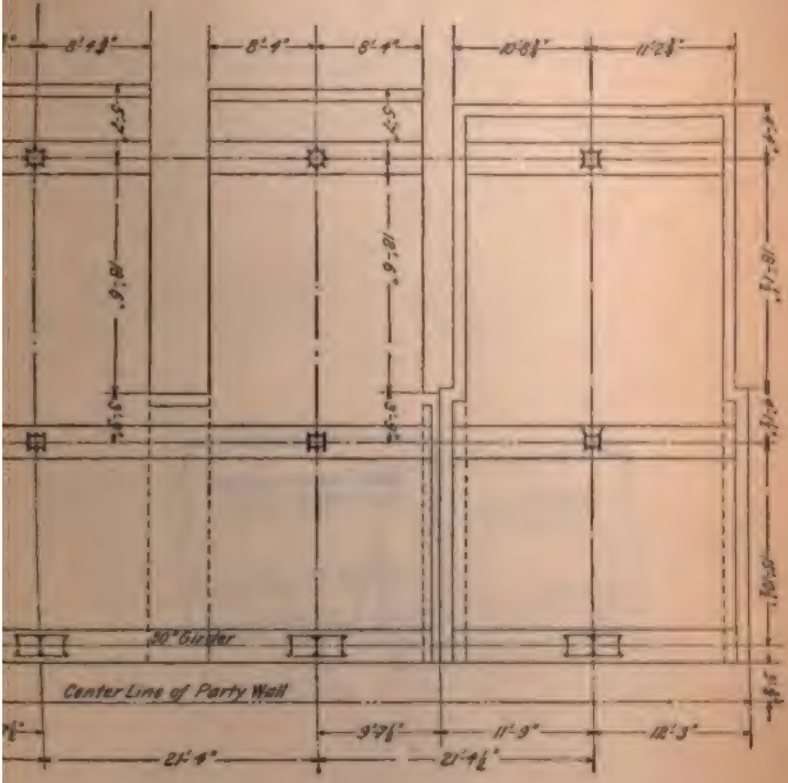
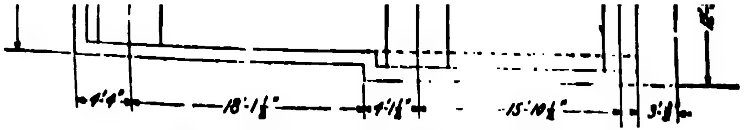
ASTOR, LENOX AND
TILDEN FOUNDATIONS

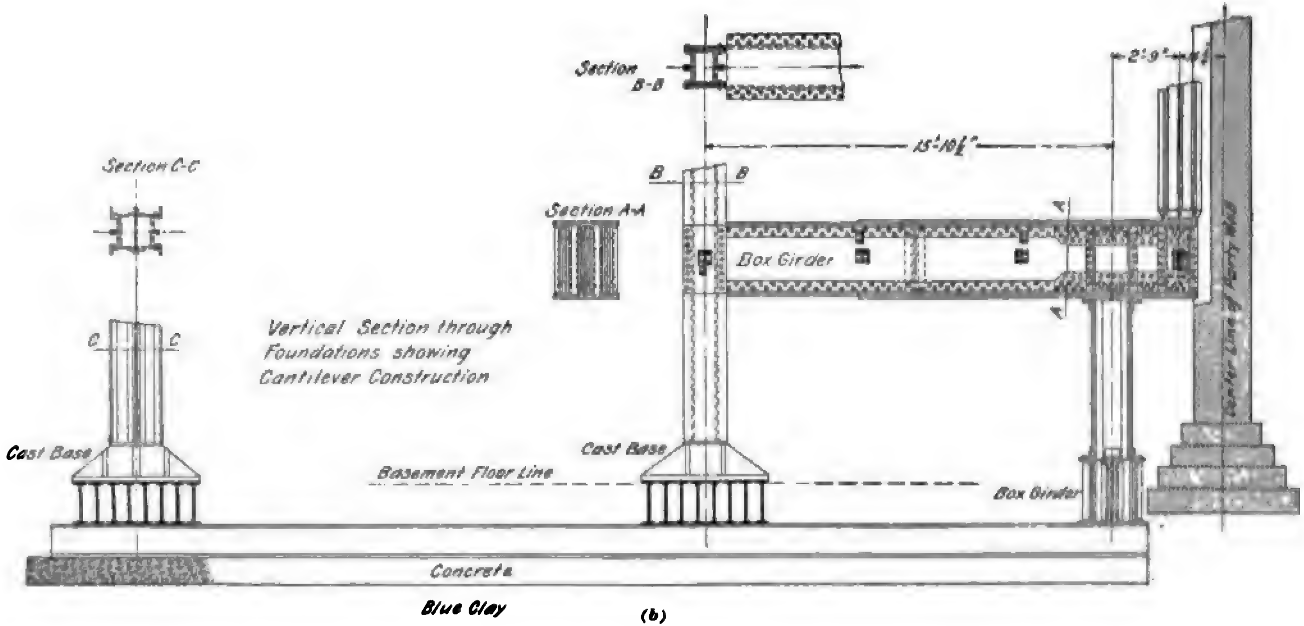


30" Girder
Center Line of Party Wall



Plan showing Location of Girders and Columns

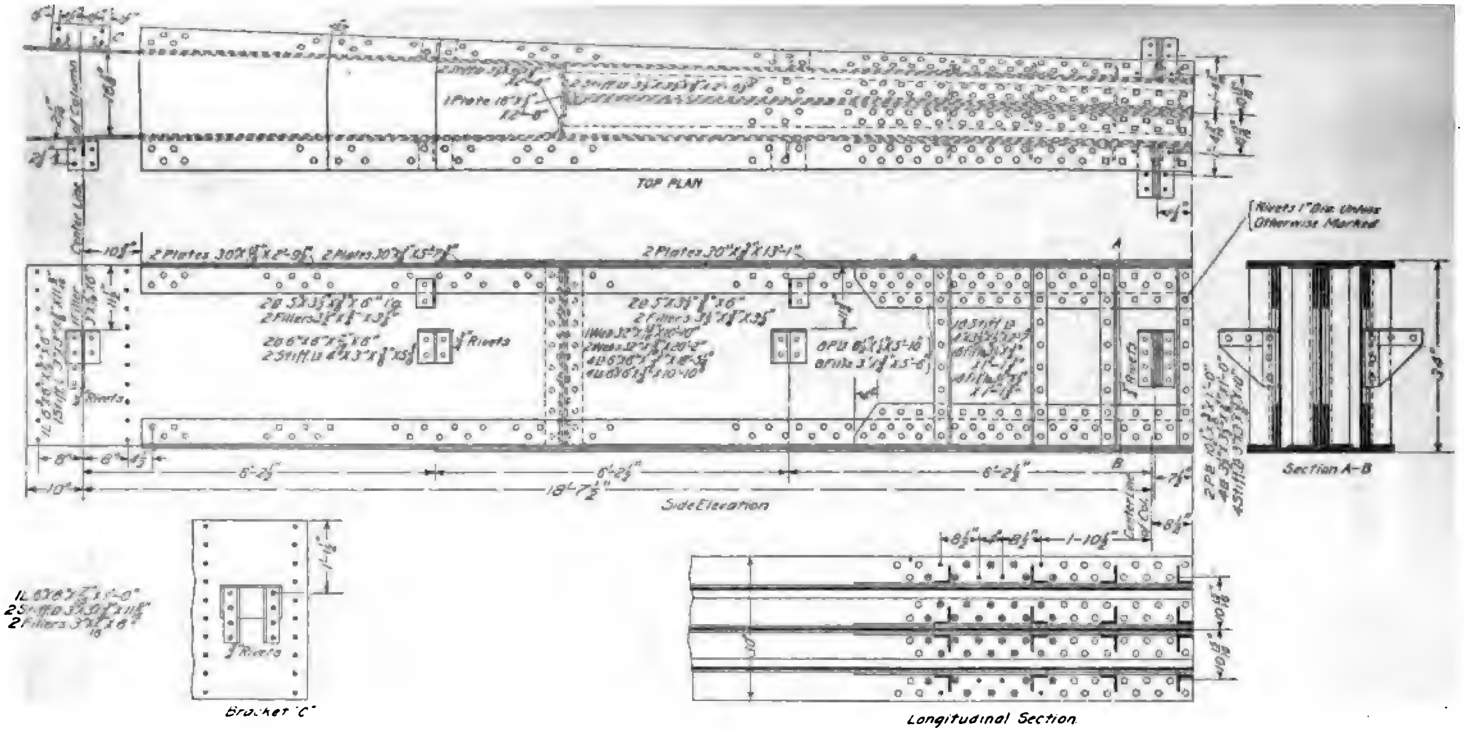
(a)



(b)

Fig. 41

ASTOR, LENOX AND
TILDEN FOUNDATIONS



TOP PLAN
Side Elevation
Section A-B
Bracket "C"
Longitudinal Section
Rivets 1" Dia. Unless Otherwise Marked

FIG. 42

proportioning the built-up girders that are commonly used for the cantilevers, the usual formulas and rules are applicable for determining the thickness of the web-plate, the flange area, the length of the flange plates, and the rivet spacing. The greatest care must be exercised in both the selection of the material and in the inspection of the workmanship of these important structural members, for their failure will ordinarily mean the destruction of the entire building. Owing to the fact that the steelwork of the cantilever construction is in a location where more than ordinary corrosion may take place with little possibility of its discovery or prevention, the steelwork should be designed with a factor of safety of from 5 to 6 as compared with the usual factors of safety of steelwork of from 3 to 4. Such a procedure would be a wise precaution and would allow for considerable deterioration without dangerously affecting the safe strength of the foundation.

RETAINING WALLS

DEFINITIONS

1. Retaining walls are employed to hold the face of a bank in position—preventing it from sliding and caving. Specifically, a retaining wall is one built to hold in place a filling or backing of earth deposited behind it after the wall has been built.

Wharf walls are masonry walls built along the water front. They sustain a pressure of earth from the back and take the wash of the water at the face. They are often required to support building walls, as in the case of ferry houses and wharf sheds, in which case they are subjected to considerable increase in pressure due to heavy cargoes in the process of loading and unloading.

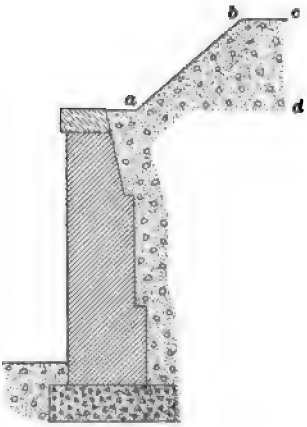


FIG. 1

The terms **filling** and **backing** are usually understood to mean loose earth, gravel, or sand dumped to fill in an excavation or back of a retaining wall.

A **face wall** is the same as a retaining wall with the exception that the term usually implies a wall built against the vertical face of natural earth formed by an excavation. The earth, being natural, is firm and solid and is not as likely to slide and cave as filling.

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Surcharge is the portion of an earth embankment above the level of the top of the retaining wall. The earth shown in section in *a b c d* in Fig. 1 is called the surcharge.

Buttresses are piers of masonry built at intervals on the face of the wall, as *a*, *a*, Fig. 2. They are usually thicker at the bottom than at the top and are employed to reenforce the wall by adding to its stability and to prevent bulging, but are little used.

Counterforts are projections or buttresses placed at the back of the wall, to supply additional weight in order to resist the overturning of the wall. Their utility is doubtful.

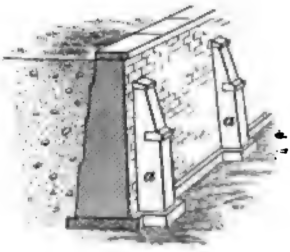


FIG. 2

Land ties are tension bars or wire-rope guys secured to the back of the retaining wall and carried through the earth to masonry, large stones, plates, or timbers embedded in the earth or filling, to which they are fastened. Any stone, plate, or timber used for this purpose is called a *dead man*. Land ties are never used in new work of good construction; they are a make-shift used principally to secure an old wall that shows signs of failure. Even for this purpose their usefulness is questionable.

The **angle of repose** is the natural slope that a material will assume when it is loosely piled; the term is also given to the angle of inclination assumed by a material when sliding and crumbling ceases.

The **coefficient of friction** is a value expressing the resistance to sliding between two particular materials; it is really the ratio between the pressure or weight on the surfaces and the force necessary to produce sliding.

STABILITY

CAUSES OF FAILURE

2. Retaining walls may fail in three ways—by *overturning*, by *bulging* in a vertical plane, and by *sliding* laterally. The **overturning** of a retaining wall, the most likely cause of failure, occurs whenever the pressure of the earth is greater than the stability of the wall. Fig. 3 (*a*) illustrates the tendency of the wall to turn about the point *c*; the resistance that the wall offers to this tendency is directly proportional to the weight of the wall and the width of its base. This is more clearly explained by reference to view (*b*), where the pressure of the earth is represented by *P*, its direction shown by the arrow, and its *point of application*, or the location at which it acts against the wall, is at *a*. It is known that the moment of any force about a point is equal to the product of the amount of the force and the perpendicular distance between the line of action of the force and the point around which it turns or tends to turn. The force *P* tends to turn the retaining wall about the point *c* and therefore exerts a moment about *c* equal to *P* multiplied by the distance *c d*. This is opposed by a countermoment about *c* equal to the weight of the wall *W* acting through the lever arm *c e*, which is the perpendicular distance from the point *c* to a vertical line passing through the center of gravity of the section of the wall. Here, then, the moment opposing the overturning tendency of the earth

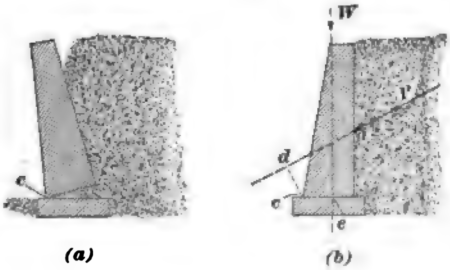


FIG. 3

equals W multiplied by the distance ce. Co
order that any retaining wall may be in equ
retically, $W \times ce$ must equal $P \times cd$.

3. The **bulging** of a retaining wall, as sh
will occur when the top and bottom of the w
place, as, for instance, in a cellar wall or in the

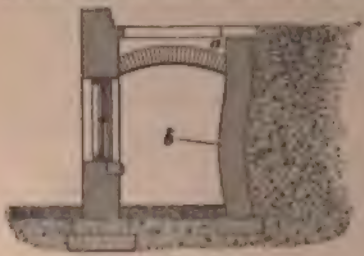


FIG. 4

supported at th
abutting agair
shown at a. B
to occur when
the point of
usually being
third the hei
footing, as
sometimes ca
ing the filling against a *green wall*, that is, or
cement or mortar has not had time to set prop

A slight bulging in a retaining wall is
matter, and need not be considered dange
exceeds $\frac{1}{4}$ inch for each foot in thickness of
point where the greatest bulging occurs.

4. **Sliding** occurs where the wall is suff
and wide at the base to prevent overturnin
owing to the nature of the
soil or the design of the
footings, the friction be-
tween the base and either
the footing or the soil is
not sufficient to prevent a
lateral movement of the

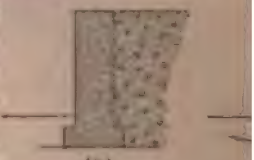


FIG. 5

According to its definition, the coefficient of friction expresses the ratio between the horizontal force required to produce sliding between the two surfaces and the weight or perpendicular pressure. Thus, under like conditions, the coefficient of friction on masonry is always the same no matter what the weight or area. To determine the resistance of a retaining wall to sliding, Table I will be found useful.

TABLE I

COEFFICIENT OF FRICTION BETWEEN MASONRY, AND EARTH AND MASONRY

Materials	Coefficient
Masonry and brickwork, dry	.65
Masonry and brickwork with wet mortar . . .	.47
Masonry and brickwork with slightly damp mortar	.74
Masonry on dry clay	.51
Masonry on moist clay	.33

By multiplying the values given in this table by the weight of the masonry in the wall, the horizontal force necessary to slide the wall laterally may be obtained; for instance, assume that a brick retaining wall rests on a wet-clay foundation. The coefficient of friction from the table is .33, and if the wall weighs 2,000 pounds per unit of length, that is, for each section of the wall 1 foot long, the force necessary to slide this portion of the wall will equal 2,000 × .33 = 660 pounds.

EXAMPLE.—Assume that the resultant horizontal pressure from the earth back of a retaining wall weighing 2½ tons per running foot and resting on dry-clay soil, is equal to 1,200 pounds; will the wall be secure against sliding and what factor of safety will it possess?

SOLUTION.—The resistance of the wall to sliding equals 5,000 × .51 = 2,550 lb. Since the horizontal pressure against the wall is equal to 1,200 lb., the wall will have a factor of safety against sliding of 2,550 lb. ÷ 1,200 = 2.125. Ans.

5. In order to provide greater resistance to sliding than is obtained by the frictional resistance between the masonry

and the soil at a level bed, it is sometimes necessary to incline the bottom surface of the footing inwards, as shown in Fig. 5 (*b*). By doing this the wall must, theoretically, be pushed up an inclined plane, as at *c d*, and the entire wall thus lifted before it can be moved laterally.

When the bed of the footing of a retaining wall slopes downwards toward the backing or filling, the angle of the slope must never approach the *angle of friction*, that is, the slope at which the weight of the masonry or footing acting on the inclined plane would just overcome the frictional resistance. It is necessary to observe this precaution because otherwise the wall will not retain its stability during construction. The angle of friction of different masonry materials is given in Table II.

TABLE II

APPROXIMATE ANGLE OF FRICTION BETWEEN MASONRY, AND EARTH AND MASONRY

Materials	Angle of Friction Degrees
Masonry and brickwork, dry	33
Masonry and brickwork laid in wet mortar . .	25
Masonry and brickwork laid in damp mortar .	36
Masonry built on dry clay	27
Masonry built on moist clay	18

6. Where a retaining wall rests on a sloping footing, as in Fig. 6, and there is no filling, it will be prevented from sliding down the slope *a b* by the friction between the wall and the footing, provided that the angle of the slope is less than the angle of friction between the wall and the footing. In the figure, the vertical line *W*, representing the weight of the wall, may be regarded as the hypotenuse of a right triangle, in which its components *c d* and *c d* representing, respectively, the pressure normal to the slope of the plane and the amount of the downward tendency of the wall to slide on the plane *a b*, constitute the other two sides.

The length of the line cd varies with the sine of the angle x and therefore the tendency of the wall to slide down the plane equals $W \sin x$. This must be the frictional resistance between the wall and the concrete footing in order that the wall may not move on the slope, or inclined surface, of the footing. It was shown in Art. 4 that the frictional resistance of a body to sliding is equal to the weight of the body multiplied by the coefficient of friction; this is equivalent to saying that the frictional resistance of a body on any plane is equal to the normal, or perpendicular, pressure to the plane multiplied by the coefficient of friction. From Fig. 6 it is evident that the pressure normal to the slope $a\,b$ is equal to $e\,d$, or, $W \cos x$. If, then, the coefficient of friction is represented by c, the frictional resistance of the wall on the plane will be expressed by $W \cos x \times c$; then, by equating, the following expression may be obtained: $W \sin x = W \cos x \times c$. By cancelation, $\sin x = \cos x \times c$, and by transposing, $c = \frac{\sin x}{\cos x}$. But the value $\frac{\sin x}{\cos x}$ is equal to the tangent of the angle x; hence, in order that a retaining wall built on a sloping bed may be just on the point of sliding downwards, the following condition must be fulfilled:

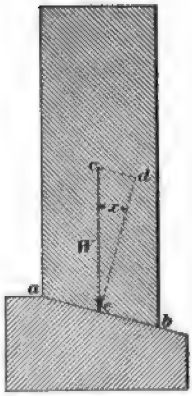


FIG. 6

$$c = \tan x \qquad (1)$$

7. This discussion assumes that the wall has just been built and that the filling has not been put in place; when the filling is in place a similar formula is necessary to express the security of a wall against sliding. In Fig. 7 is represented a retaining wall resting on a sloping base of concrete. The force P is the resultant of the pressure of the earth, which, in this case, is assumed as acting in a direction parallel with the slope of repose of the material composing the filling. The weight of the wall acts on the plane $a\,b$ vertically and its resultant W may be considered as being

coincident with a line passing through the center of gravity of the wall. The third force of the system is the force S, or the tendency of the wall to slide down the inclined plane $a\,b$. It is assumed that the force S is applied at a common point with the other two forces, and, consequently, the resultant of the system of forces may readily be obtained by drawing the polygon of forces described by the lines w, p, s, and P_1; the line P_1 represents the resultant of the three forces tending to push the wall up the slope. In order that P_1 shall not push the wall up the slope $a\,b$, the frictional resistance must be equal to that component of P_1 which is parallel with the plane or sloping bed of the footing. This component is S_1 and it varies with the sine of the angle x_1, so that

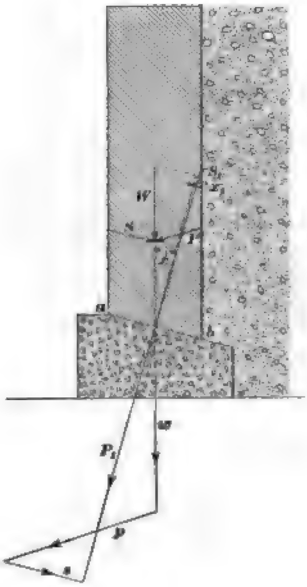


FIG. 7

$$P_1 \sin x_1 = W \cos x \times c \qquad (2)$$

This equation expresses the condition that exists when the pressure of the earth is just equal, in its tendency to push the wall up the incline, to the frictional resistance of the wall. Hence, a retaining wall will be secure against sliding on an inclined bed when $W \cos x \times c$ exceeds $P_1 \sin x_1$; that is, when the product of the weight of the wall, the coefficient of friction, and the cosine of the angle that a vertical line makes with a line normal to the slope of the bed exceeds the product of the resultant pressure on the wall and the sine of the angle formed by the line of action of the resultant pressure and a line normal to the slope. A retaining wall will always be secure against sliding when the angle included between the line of action of the resultant pressure and a line normal to the slope is less than the angle of friction, which is the condition that should be obtained.

THEORY OF STABILITY

8. Many theories relating to the **stability** of retaining walls have been evolved by eminent physicists, and, though interesting, they have been proved by practice and experience to be unreliable. All of them are founded on the same three assumptions; namely, that the line of rupture of the material composing the backing is a plane, that the point of application is known, and that the direction of the thrust may be determined. One of these assumptions, the first, is known by observation to be false, while the others, though they seem reasonable, have not been conclusively proved.

In the first assumption, the earth at the back of the wall is considered as tending to slide along an oblique plane *a b*, Fig. 8 (*a*). There is but one character of soil, clean dry

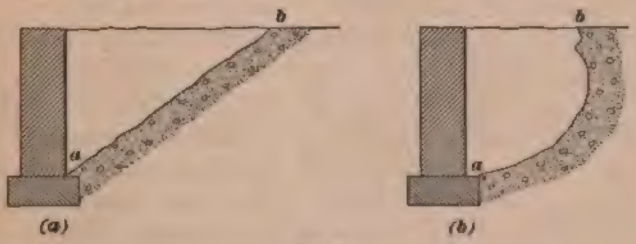


FIG. 8

sand, that will realize this theoretical condition. All other soils possess considerable tenacity and their line of rupture will approach the line *a b*, as shown in Fig. 8 (*b*).

The angle that the assumed plane of rupture makes with the horizontal is called the angle of repose of the earth or material. This angle of repose, or natural slope, varies greatly with the different soils, and also with soils of the same nature containing different amounts of moisture. Damp sand or earth will stand when the slope has a rise of one to a horizontal run of one, but the best practice is to consider the angle of repose for all earths and soils as 33° 41′; that is, one and one-half of horizontal run to one of rise. This is the natural slope of dry earth, and as the pressure against a retaining wall is greater with dry

material than with wet, provided that the latter does not possess such a degree of fluidity as to produce hydrostatic pressure, this angle is usually assumed.

The second assumption, that the point of application of the thrust is at a distance of one-third the height from the base, is equivalent to considering the earth backing as a fluid, devoid of tenacity and friction. That the center of pressure of a fluid against a vertical wall is at a distance of one-third the height from the bottom is evident from the fact that the pressure varies uniformly with the depth. For instance, in a reservoir 10 feet high, if the water weighs .434 pound for each foot in depth, the pressure at the surface of the water is zero; at 5 feet from the top the pressure is $.434 \times 5 = 2.170$ pounds; while at the bottom the pressure is $.434 \times 10 = 4.34$ pounds. If a triangle, Fig. 9, is drawn to some scale so that *a b* equals the depth of the water and *b c* equals the pressure at the bottom, the pressure at any point may be found by measuring the horizontal dotted lines, or *abscissas*, as they are termed in analytic geometry. The triangle *a b c* then diagrammatically represents the pressure of the water against the wall of the reservoir, and as the center of gravity of a triangle is always in a line parallel with the base at a distance of one-third the height from the base, as in this case at *x y*, the total pressure may be considered as being concentrated at this height.

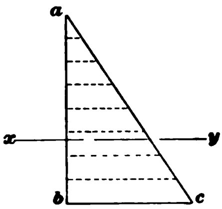


FIG. 9

Since the nature of earth varies so materially from that of water, the second assumption is far from correct, though it is probably as nearly so as any that can be made where a specific value must be general in its application.

The third assumption, regarding the direction of the thrust of the earth, varies with the several theories regarding the stability of retaining walls. In the theory advanced by Coulomb, the direction of the earth's pressure is assumed as perpendicular to the face of the wall; this assumption disregards the friction of the earth against the face of

the wall, and therefore cannot be correct. The physicist Weyrauch determines the angle that the line of pressure makes with the wall by a complicated trigonometric formula deduced from an elaborate theory that has little practical value, for, in opposition to all practical results, it gives a greater pressure when the wall is inclined backwards toward the natural slope than when it is straight or leans forwards; it also neglects the vertical component of the earth's pressure, which certainly exists.

METHODS FOR DETERMINING STABILITY

ANALYTICAL METHODS

9. Coulomb's Theory.—The first theory for determining the stability of retaining walls, and the one on which most of the other theories, since evolved, are based, is known as **Coulomb's theory.** This is principally interesting as a study, and usually gives results that err on the side of safety, for walls that it demonstrates to be about to fall seem to be perfectly stable, and apparently possess a factor of safety of at least 2. Coulomb assumes that the earth of an embankment held in place by a retaining wall tends to slide along a straight line or plane, that the center of pressure of the earth acts at a point one-third of the height of the wall from the base, and that the line of pressure acts perpendicular to the face of the wall. From these assumptions he determines, by reasoning and calculations, that the maximum pressure on a vertical wall, as in Fig. 10, is not due to the weight of the triangle *c a b*, but is caused by the wedge-shaped prism of earth *c a d* included between the vertical face of the wall and the line *a d* that bisects the angle *c a b*. The line *a b* forms an angle with the horizontal equal to the angle of repose of the material. The line *a d* is called the

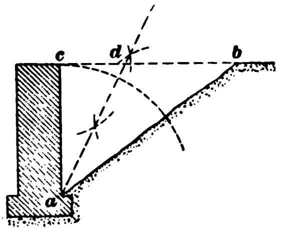


FIG. 10

slope of maximum pressure, and the prism, a section of which is represented by the triangle *c a d*, is called the ***prism of maximum pressure.***

In Coulomb's theory, the earth is considered as a liquid and the total horizontal hydrostatic pressure on any vertical wall of unit length is equal to one-half of $w_1 h^2$, in which w_1 equals the assumed weight of the material per unit of volume and h equals the height of the wall. In the case of the prism of earth at the back of the retaining wall, the thrust, by Coulomb's theory, is equal to the thrust produced by a liquid whose unit weight w_1 is equal to $w \tan^2 \frac{1}{2} c\,a\,b$; that is, the material of which the filling or backing is composed is assumed to be a liquid having a unit weight w_1 equal to the actual unit weight w of the material multiplied by the square of the tangent of one-half the angle $c\,a\,b$, Fig. 10. By substituting this value for w_1 in the expression $\frac{1}{2} w_1 h^2$, the horizontal pressure on a wall of unit length is

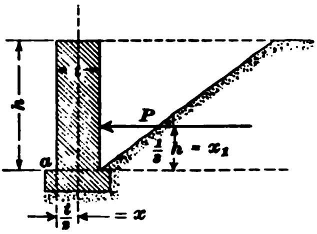


FIG. 11

$$P = \tfrac{1}{2} w h^2 \tan^2 \tfrac{1}{2} c\,a\,b \qquad (3)$$

This formula may be expressed as follows:

Rule.—*The horizontal pressure on a vertical wall of unit length is equal to one-half the weight of the filling per unit of volume times the square of the height of the wall, in feet, multiplied by the square of the tangent of one-half the angle between the back of the wall and the natural slope of the material.*

When the horizontal pressure P, in Fig. 11, is known, the thickness of a retaining wall having a vertical face and back can be readily evolved from the principles of Coulomb, in which $W_1 x$ must equal $P x_1$, or the moment of the weight of the wall about a certain point a must equal the moment of the pressure of the earth about the same point, in order to have a stable wall. As W_1 in this equation equals the

entire weight of a unit length of the wall and x equals one-half the thickness of the base, $W_1 x = W t h \times \frac{t}{2}$, or $\frac{W t^2 h}{2}$, in which W equals the weight of the masonry in the wall per unit of volume, usually taken in cubic feet; t equals the thickness of the wall, in feet; and h equals the height, in feet. In the expression $P x_1$, according to the assumptions of the theory, x_1 equals one-third the height of the wall, as shown in Fig. 11, and then by substituting the value of P obtained by formula **3**, $P x_1 = \frac{1}{3} h \times \frac{1}{2} w h^2 \tan^2 \frac{1}{2} c a b$, or $P x_1 = \frac{1}{6} w h^3 \tan^2 \frac{1}{2} c a b$. By equating this statement with the one preceding, we get $\frac{W t^2 h}{2} = \frac{1}{6} w h^3 \tan^2 \frac{1}{2} c a b$, which by simplification and transposition becomes $t^2 = \frac{w h^2 \tan^2 \frac{1}{2} c a b}{3 W}$,

or $$t = h \tan \tfrac{1}{2} c a b \sqrt{\frac{w}{3 W}} \qquad (4)$$

This formula, which gives the thickness t of a retaining wall when filled in level with the top, may be expressed by the following rule:

Rule.—*The mean thickness of a retaining wall, in feet, may be obtained by multiplying the height of the wall, in feet, by the tangent of one-half the angle included between the back of the wall and the natural slope of the material, and multiplying this product by the square root of the quotient found by dividing the weight per cubic foot of the earth filling by three times the weight per cubic foot of the masonry.*

Example.—The retaining wall along one side of a factory yard is built of rubble masonry laid up in cement mortar. The distance from the top of the footing course to the top of the earth filling, which is level with the top of the wall, is 10 feet. Provided that the earth filling has the usual slope of repose of $1\frac{1}{2}$ to 1, what will be the mean thickness of the wall by Coulomb's rule?

Solution.—The weights per cubic foot of earth and rubble masonry are found to equal 85 and 150 lb., respectively.

The angle of repose of the earth has a horizontal run of $1\frac{1}{2}$ to a rise of 1. If in a right triangle the two sides represent the rise and the run, respectively, the tangent of the angle opposite the rise is $1 \div 1\frac{1}{2}$

= .66666. From trigonometrical tables, the angle corresponding to the natural tangent .66666 is 33° 41′, which is the value of the angle included between the horizontal and the natural slope of the material, while the angle $c\,a\,b$, Fig. 10, required by formula 4 is the difference between 90°, or the right angle included between the horizontal and the vertical back of the retaining wall, and the angle 33° 41′. The angle $c\,a\,b$ then is 90° − 33° 41′ = 56° 19′, and since the formula requires the tangent of one-half the angle, the value required equals tangent ½ of 56° 19′ or tan 28° 9′ 30″ = .53526. By substituting the values of h, w, and w_1, given in the example, and the value of tan ½ $c\,a\,b$ just obtained, in formula 4,

$$t = 10 \times .53526\sqrt{\frac{85}{3 \times 150}} = 2.33 \text{ ft.} \quad \text{Ans.}$$

However, it is seldom that a retaining wall is made the same thickness at the top as at the bottom, especially a wall as high as the one in the problem. It is usual to make it of a thickness at the top equal to $\frac{1}{10}$ to $\frac{1}{8}$ the height, and where the wall is battered, as is usual in practice, the section of the wall becomes a trapezoid. The sectional area of a trapezoidal wall is obtained by the equation $A = h\frac{a+b}{2}$. By transposition, the value of a, or the width of the wall at the bottom, is equal to the equation $a = \frac{2A}{h} - b$. The value of A, or the sectional area of the retaining wall, equals the height h, which is known from the conditions of the problem, multiplied by the mean thickness of the wall found by calculation. Thus, $A = h \times t = 10 \times 2.33 = 23.3$ sq. ft. The value of b, or the thickness at the top, is made equal to $\frac{1}{10}h$ or 1 ft., for it is assumed that coping stones 15 in. wide are to be used. By substitution in the equation, therefore, $a = \frac{2 \times 23.3}{10} - 1$, or 3.66 ft. The wall, to fulfil the requirements, should be designed with a thickness at the base equal to 3.66 ft. or nearly 3 ft. 8 in., and a thickness at the top equal to 1 ft.; the face and back, respectively, of the wall will be battered and stepped as shown in Fig. 12. Ans.

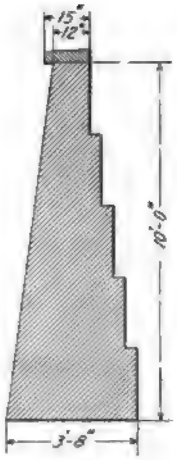


FIG. 12

10. Rankin's Theory.—The theory evolved by Rankin is somewhat similar to Coulomb's; he has determined that the resultant pressure of the filling applied horizontally and at a distance of one-third the height of the wall from the

base, is equal to the value obtained by formula **5**, when the earth is level with the retaining wall.

$$P = \frac{w h^2}{2} \times \frac{1 - \sin x}{1 + \sin x} \qquad (5)$$

in which P = horizontal resultant pressure of earth filling, in pounds;

w = weight of earth, in pounds per cubic foot;

h = height of retaining wall, in feet;

x = angle that natural slope of repose of material makes with horizontal.

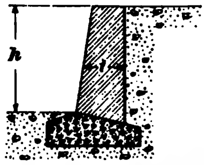
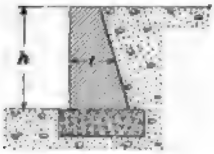
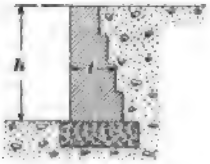
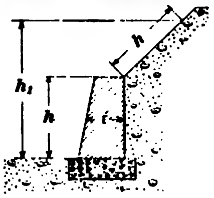
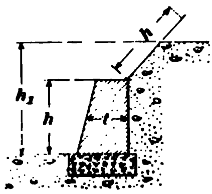
When the pressure of the earth has thus been determined by **Rankin's formula,** its moment about the outside edge of the retaining wall may be calculated and the stability of the wall analyzed by comparing this moment with the moment of the weight of the wall about the same point, which is equal to the weight multiplied by the horizontal distance from a vertical line passing through the center of gravity of the wall to the outside edge of the wall or footing.

Based on Rankin's theory, practical formulas for determining the thickness of retaining walls have been worked out. These formulas, given in Table III, are a modification of Rankin's investigations, and may be relied on to the extent of any rule or formula for the design of a structure subject to such uncertain conditions as a retaining wall.

In Table III, the value of K is the same in all the formulas, and is obtained by the formula given in Case I under the heading Remarks. This formula, as will be observed, introduces the angle x, which is the angle of repose of the filling, and in applying the formulas, Table IV, giving the angles of repose of different earths and the corresponding values of K will be found useful. It will be noted, in Cases V and VI of Table III, that the value h_1 is substituted for the height of the wall and that Case VI differs from Case V only on account of the difference in the value of h_1. In Case V, it will also be observed that the length of the slope of the embankment or surcharge is longer than the height of the wall, while in Case VI the length of

TABLE III

PRACTICAL FORMULAS FOR DETERMINING THE T OF RETAINING WALLS

Section Showing Conditions		Formulas for Thickness	
Case I		$t = Kh\sqrt{\frac{w}{W}}$	$K =$ o
Case II		$t = .86\ Kh\sqrt{\frac{w}{W}}$	batter
		$t = .80\ Kh\sqrt{\frac{w}{W}}$	batter
		$t = .74\ Kh\sqrt{\frac{w}{W}}$	batter
Case III		$t = .85\ Kh\sqrt{\frac{w}{W}}$	batter
Case IV		$t = .85\ Kh\sqrt{\frac{w}{W}}$	steppec
Case V		$t = .86\ Kh_1\sqrt{\frac{w}{W}}$	batter
		$t = .80\ Kh_1\sqrt{\frac{w}{W}}$	batter
		$t = .74\ Kh_1\sqrt{\frac{w}{W}}$	batter
Case VI		$t = .86\ Kh_1\sqrt{\frac{w}{W}}$	batter
		$t = .80\ Kh_1\sqrt{\frac{w}{W}}$	batter
		$t = .74\ Kh_1\sqrt{\frac{w}{W}}$	batter

the slope of the embankment or surcharge is less than the height of the wall. In Case V, the distance h_1 is measured vertically to a point on the natural slope, measured parallel with the slope and equal to the height of the wall from the inner top edge. In Case VI, the distance h_1 is measured vertically to the top of the surcharge. W represents the weight of a cubic foot of wall, while w equals the weight of a cubic foot of the filling at the back of the wall; t is the mean thickness of the wall, in feet.

TABLE IV

WEIGHT OF EARTHS, ANGLE OF REPOSE, AND VALUE OF K

Description of Materials	Pounds per Cubic Foot w	Angle of Repose x	K equals $.7 \tan \frac{90 - x}{2}$
Clay, dry	100	45°	.29
Clay, wet	125	17°	.52
Earth, dry loose	85	32°	.39
Earth, dry rammed	110	37°	.35
Earth, very compact	115	55°	.22
Earth, damp	100	29°	.41
Earth, wet rammed	125	27°	.43
Gravel, dry	110	32°	.39
Gravel, wet	125	24°	.45
Gravel, with sand	115	26°	.44
Loam, dry	100	40°	.33
Loam, wet	130	17°	.52
Pebbles, round	110	23°	.46
Pebbles, sharp	110	45°	.29
Sand, dry	112	32°	.39
Sand, damp	115	26°	.44
Sand, wet	125	24°	.45
Stones, sharp broken	100	38°	.34

EXAMPLE 1.—What will be the thickness required for a retaining wall of the section shown in Case II of Table III, when the wall is

6 feet high and has a batter of 1 inch to 1 foot? The wall is of limestone masonry and the filling is wet loam.

SOLUTION.—From Table III, the formula for obtaining the mean thickness is $t = .86\,K'h\sqrt{\frac{w}{W}}$. From tables giving the weights of masonry, the weight of limestone masonry is found to be 165 lb. per cu. ft. The value of K' from Table IV is .52 and the weight of wet loam is 130 lb.; substituting the values in the formula,

$$t = .86 \times .52 \times 6\sqrt{\frac{130}{165}} = 2.38 \text{ ft., or 2 ft. } 4\tfrac{1}{2} \text{ in. approx.}$$

Since this is the mean thickness of the wall and the batter is 1 in. to 1 ft. the wall will be 2 ft. 7½ in. thick at the bottom and 2 ft. 1½ in. at the top. Ans.

EXAMPLE 2.—In a surcharged wall of the section shown in Case V of Table III, the height of the wall is 8 feet and the batter is 1½ inches to 1 foot. The level of the bank is 10 feet above the top of the wall. If the wall is of sandstone masonry and the filling is dry earth rammed: (*a*) what mean thickness of wall will be necessary to hold the backing in place? (*b*) what will be the thickness at the top and bottom?

SOLUTION.—(*a*) The weight of sandstone masonry is 145 lb. and, from Table IV, the weight of dry earth rammed is 110 lb. and the value of K' is .35. The value of h_1 may be determined as follows: Draw a horizontal line at the top of the wall and from a point on the slope at a distance from the top of the wall equal to h or the height of the wall, draw a perpendicular to the horizontal line. The length of this perpendicular added to the height of the wall gives the value of h_1 to be used in the formula. The angle between the slope of the surcharge and the horizontal is always considered as being equal to the angle of repose of the material composing the backing. In this case the angle of repose is 37°, and, by scaling, we find the length of h_1 to be 12 ft. 9½ in., or 12.8 ft. Substituting the values in the formula $t = .80\,K'h_1\sqrt{\frac{w}{W}}$, gives

$$t = .80 \times .35 \times 12.8\sqrt{\frac{110}{145}} = 3.12 \text{ ft.}$$

or 3 ft. 1 in., mean thickness. Ans.

(*b*) The wall is 8 ft. high and the batter 1½ in. to 1 ft.; consequently, the difference in thickness between the top and bottom will be $8 \times 1\frac{1}{2} = 12$ in., or the thickness at the top will be 6 in. less, and at the bottom 6 in. more, than the mean thickness. Hence, the thickness at the top is 3 ft. 1 in. − 6 in. = 2 ft. 7 in.; thickness at bottom is 3 ft. 1 in. + 6 in. = 3 ft. 7 in. Ans.

EXAMPLES FOR PRACTICE

1. Employ Table III in determining the thickness to the nearest inch at the top and bottom of a retaining wall having a vertical face with the back stepped 4 inches in 2 feet and built of brick laid in cement mortar and weighing 130 pounds per cubic foot; the filling is gravel with sand, made level with the top of the wall, the height of which is 8 ft.

Ans. {Top, 2 ft. 2 in.; Bottom, 3 ft. 6 in.}

2. Calculate, by means of Table III, the top and bottom thickness to the nearest inch of a retaining wall composed of granite rubble masonry weighing 150 pounds per cubic foot and supporting a backing of damp earth. The wall is 7 feet high, having a batter on the face of 1 inch to 1 foot, and the level of the backing is 2 feet above the top of the wall.

Ans. {Top, 2 ft. 4 in.; Bottom, 2 ft. 11 in.}

3. What will be the thickness to the nearest inch at the top and bottom of a retaining wall 10 feet high, built of sandstone, that supports a backing of wet clay level with the top of the wall? The weight of sandstone masonry may be taken at 145 pounds per cubic foot. Use Coulomb's theory in making the calculation, and consider the face of the wall as being battered 2 inches in every foot.

Ans. {Top, 3 ft. 2 in.; Bottom, 4 ft. 10 in.}

4. It was found necessary, in order to provide headroom for a set of boilers, to excavate the basement of a building for 6 feet below the cellar bottom. The excavation extended over a considerable portion of the basement but did not approach nearer than 10 feet to the footings of the building. What thickness of brick wall would be required for facing this excavation, provided it was laid in cement mortar, its weight being 130 pounds per cubic foot? The face and back of the wall are straight and the earth in the basement is very compact.

Ans. 1 ft. 3 in.

5. According to Rankin's formula **5**, what would be the horizontal pressure on a wall 9 feet high holding in place a filling of sharp broken stones, level with the top of the wall? Ans. 963 lb.

GRAPHICAL METHOD

11. The **graphical method** for determining the stability of a retaining wall is based on Coulomb's and Moseley's theories. It approaches the true conditions that exist more nearly than the analytical method, for it takes into account the friction between the earth and the back of the wall.

This friction is considerable when the back of the wall is stepped or battered, as shown in Fig. 13 (a) and (b), and adds greatly to the stability of the wall.

The friction between the earth and the back of the wall, tending to act in a vertical direction, is assumed to influence the pressure P, Fig. 14, acting perpendicular to the wall and at a distance of one-third the height from the base, so that the resultant pressure will be P_1. The pressure P_1 is assumed to act in a plane at an angle of 33° 41′ to the plane of P, which, as previously explained, is the angle of repose of dry earth.

FIG. 13

The amount of this pressure P_1 may be determined when the value of P is known, by the principle known as the **resolution of forces.** For instance, in Fig. 14, the amount of the pressure P is laid off from i to e to scale. The point being thus located, the line ef is drawn upwards at right angles to ie until it intersects at f the line representing the pressure P_1, which had been previously drawn at an angle of 33° 41′ with the line representing the pressure P, or ie. By measuring the line fi by the same scale with which ie was measured, the amount of the pressure P_1 may be determined. In this case, the line fi, which represents the pressure along the plane P_1, is found by measurement to be 2,400 pounds.

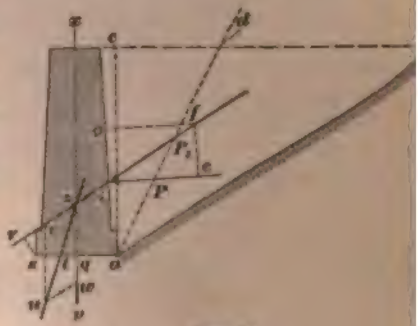


FIG. 14

The lines ie, ef, fg, and gi make a parallelogram of forces in which ie equals P, the theoretical pressure of the earth

perpendicular to the wall; fe equals the downward force representing the friction of the earth against the back of the wall; and fi equals the resultant of both these forces, and the one to be considered in determining the stability of the wall. As the line fi represents the pressure P_1 accurately to scale, and the secant of the angle fie is equal to $\frac{P_1}{P}$, it follows that the pressure P_1 may also be determined by the equation $P_1 = P \sec fie$.

According to formula **3**, $P = \frac{1}{2} w h^2 \tan^2 \frac{1}{2} cab$, which may be simplified by introducing W instead of $\frac{1}{2} w h^2 \tan \frac{1}{2} cab$, since the latter expression equals W, or the weight of the prism of earth acd. In the equation, w and h equal, respectively, the unit weight of the earth backing and the height of the retaining wall, as described in Art. **9.**

By substituting W in the equation, $P = \frac{1}{2} w h^2 \tan^2 \frac{1}{2} cab$, it becomes $P = W \tan \frac{1}{2} cab$, which may be further simplified by introducing the distance cd, which is equal to $h \tan \frac{1}{2} cab$. Multiplying the expression $W \tan \frac{1}{2} cab$ by $\frac{h}{h}$,

$$P = \frac{W c d}{h} \qquad (6)$$

It is necessary to find the distance cd, which may be calculated, or if not convenient to use trigonometrical functions, it may be obtained by making a scale drawing as in Fig. 14, in which the line ad bisects the angle cab included between the vertical and the plane of the slope of repose of the material; from this figure the distance cd may be measured. The pressure P is then found by formula **6**, which may be expressed by the following rule:

Rule.—*The maximum pressure P, in pounds, acting perpendicular to the back of a retaining wall and at a point one-third of the height from the base, is equal to the product of the weight of the prism of maximum pressure, in pounds, and the distance c d, in feet, divided by the height of the wall, in feet.*

When the pressure P has been obtained by either formula **5** or **6**, the stability of the retaining wall may be determined

by the graphical method illustrated in Fig. 14. If the back is stepped as in Fig. 13 (*a*), the line *e f*, Fig. 14, instead of being drawn as described, will be made parallel with a line passing through the edges of the steps, as *a b*, Fig. 13. The amount of the pressure P_1 is found by scaling the line *i f*. Extend the line *f i*, or the pressure P_1, downwards indefinitely and draw the vertical line *x y* through the center of gravity of the wall section. If the wall is unsymmetrical, the line *x y* may be located by cutting a scale section of the wall from cardboard, and balancing it on the edge of a knife, or straightedge, or its location may be determined by calculation. The intersection of the line *f i* extended, with the vertical line *x y*, gives the point *z*; from this point measure off the distance *z w*, by any scale equal to the weight of the retaining wall. In figuring the weight of the retaining wall, a unit length, usually 1 foot, of the wall is considered. From the point *z* on the line *f i* extended, lay off the amount of the pressure P_1 to the same scale used in measuring *z w* and thus locate the point *v*. From *w*, draw *w u* parallel with *f i* extended, and from *v* draw *v u* parallel with *x y*, in this way obtaining the point *u*. Through the points *z* and *u*, draw the line *z u* intersecting the base of the wall at *t*. Then, with good masonry and unyielding soil, the wall is stable if the resultant pressure represented by the line *z u* intersects the base line so that *s t* is equal to at least one-fifth of *s a*, and provided the product of the force P_1 by the lever arm *r s* does not exceed the product of the weight of the wall by the lever arm *s q*. The quotient obtained by dividing the latter by the former gives the factor of safety of the wall against overturning. A retaining wall that is safe against overturning according to this method is, under normal conditions, safe from failure by either bulging or sliding.

EXAMPLE.—It is required to ascertain the stability of the rubble retaining wall shown in Fig. 15, the earth having a natural slope of 1½ to a rise of 1.

SOLUTION.—It is assumed that a scale drawing, as shown in Fig. 15, has been made, from which the weight of the wall 1 ft. in length may be calculated. It is considered in this instance that the small

projecting portion q of the cap, or coping, is equal, in area, to the portion cut from the top of the coping to make the bevel or wash; consequently, the weight of the section $m\,n\,o\,p$ is the weight of the top

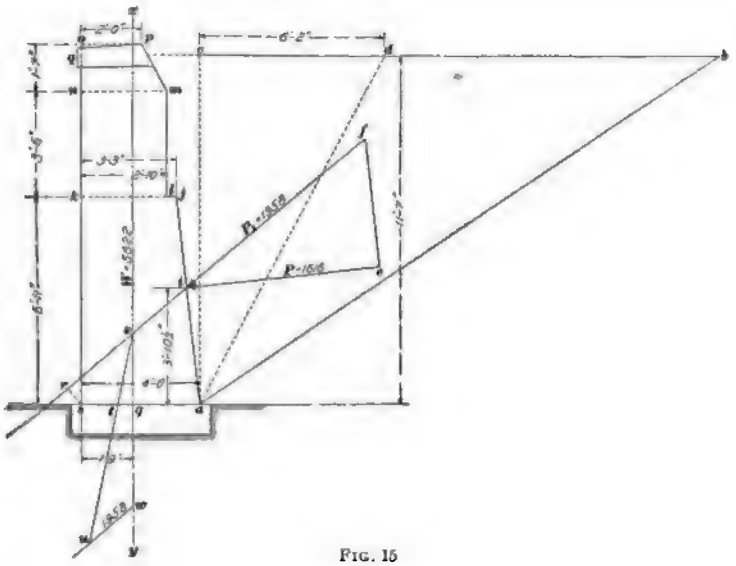


FIG. 15

portion of the wall including the weight of the small projecting piece q. The weight of rubble masonry is 150 lb. per cu. ft. Then the calculations for the weight of the wall of unit length may be arranged as follows:

$$\text{Section } a\,s\,k\,j = \frac{3.25 + 4}{2} \times 6.9167 \times 150 = 3760.95 \text{ lb.}$$

$$\text{Section } l\,k\,n\,m = 2.8333 \times 3.5 \times 150 = 1487.49 \text{ lb.}$$

$$\text{Section } m\,n\,o\,p = \frac{2.8333 + 2}{2} \times 1.5833 \times 150 = 573.96 \text{ lb.}$$

$$\text{Total weight of wall 1 foot in length} = 5822.40 \text{ lb.}$$

The weight of the wall having been ascertained, it is now necessary to determine the location of the line xy that passes through the center of gravity of the wall. In this instance, the line xy is located at a distance from the vertical face $o\,s$ equal to 1 ft. 9 in. The line $a\,b$ should be drawn at the angle of repose of the material or with a rise of 1 to a run of $1\frac{1}{2}$ and the angle $c\,a\,b$ bisected by the line $a\,d$. From the measurements thus obtained and shown in the figure, the area of the triangle $c\,a\,d$ and weight of the prism may be calculated. The area of the triangle $c\,a\,d$ is equal to $(6.1667 \times 11.5833) \div 2 = 35.715$ sq. ft., and as this prism of earth is regarded as 1 ft. in

length, the same as the wall, the cubical contents is 35.715 cu. ft. The weight of 1 cu. ft. of earth is found to equal about 85 lb.; consequently, the weight of the prism $c\,a\,d$ is $35.715 \times 85 = 3{,}036$ lb. All the data required by formula 6 are now obtained, or known from the conditions of the problem, hence the pressure P perpendicular to the back of the wall may be figured from the formula, $P = \frac{W\,c\,d}{h}$, and by substituting,

$$P = \frac{3{,}036 \times 6.1667}{11.5833} = 1{,}616 \text{ lb.}$$

Laying off the forces in the diagram, Fig. 15, to some convenient scale, the distance representing the force P of 1,616 lb. is laid off from i to e, the point i being at a distance above the base equal to one-third the height of the wall. Having located e, draw a line upwards from this point, parallel with the back of the wall $i\,j$. From the point i an indefinite line, as $i\,f$, is drawn at the angle of repose with $i\,e$, or 33° 41′, and where this line intersects the line drawn upwards from e, the point f is located; then $i\,f$ represents the force P_1, which may be determined by measuring with the scale used in laying out the diagram. In this instance, the distance $i\,f$ is found to represent about 1,958 lb. By extending the line $f\,i$, the point of intersection with the line $x\,y$ is obtained, as at z. From z, a distance equivalent to the weight of the wall is laid off downwards and the point w thus located. In this particular case, owing to the great weight of the wall, a different scale will be employed to lay off the distance $z\,w$ from that used in measuring P and P_1. From w is drawn an indefinite line downwards and parallel with the line $f\,i$ extended, and on this line is marked off a distance $w\,u$ that will represent the thrust P_1 of 1,958 lb., as previously obtained. The resultant pressure on the footing is determined by a line drawn through z and u. Since in this case the line $z\,u$ cuts the base of the wall at the point t, which is at a distance from s greater than one-fifth of the entire width of the wall, or $s\,a$, the wall is safe against overturning, provided that the moment of P_1 about s is less than the moment of the weight of the wall about the same point.

When the moments about the point s are considered, the lever arm through which P_1 acts is $s\,r$, or 8 in., while the lever arm of the weight of the wall or W is equal to the horizontal distance from the point s to a vertical line passing through the center of gravity of the section, which equals 1 ft. 9 in., or the distance $s\,q$. The moment of P_1 is $1{,}958 \times .6667 = 1{,}305$ ft.-lb., while the moment of W is $5{,}822 \times 1.75 = 10{,}189$ ft.-lb. In consequence, the wall is entirely stable and secure against overturning; in fact, it could be reduced in section to a considerable extent, though it would not be advisable, in order to follow the details of good practice, to change the width of the base materially. Ans.

It will be noticed in the solution of this problem that the point of rotation was taken at *s* and that the natural slope of repose *a b* and the line *a d* representing the plane of maximum pressure were drawn from the point *a*. This is a reasonable assumption when the footing is of concrete with a level base at *s a*.

EMPIRICAL DESIGN

12. It is not usual to place much dependence on the theoretical methods and discussions relative to the stability of retaining walls, and engineers usually follow some empirical rule that, from their experience, has proved satisfactory.

Where a retaining wall is built of good masonry or brickwork plumb on the face and back and where the filling is of the usual material, that is, earth mixed with gravel and sand moderately packed, it will be stable under ordinary conditions, provided that the base is three-tenths of the height and the earth is level with the coping.

TABLE V

THICKNESS OF RETAINING WALL AT BASE IN TERMS OF THE HEIGHT FROM BASE OF WALL TO TOP OF EARTH FILLING

Character of the Masonry	Percentage of Entire Vertical Height From Base of Wall to Top of the Earth Filling
Cut stone, or ranged rubble, lime mortar	.35
Good rubble or brickwork, lime mortar .	.40
Scabbled or partially dressed, dry rubble	.50

If the wall is battered on the face and stepped at the back, a width of base equal to four-tenths of the height will insure stability, provided that the batter does not exceed $1\frac{1}{2}$ inches per foot on the face and the stepping of the back does not exceed 4 inches for a vertical height of 2 feet 6 inches.

A wall having a base equal to one-half the height will usually be stable when constructed of the poorest masonry and under the most unfavorable conditions of soil.

Table V gives data, founded on experience rather than theory, for proportioning the width of a retaining wall at the base where it bears on the footing. The earth backing is assumed to be level with the top of the wall and to be of the usual material loosely deposited, as it would be if dumped from carts or cars.

13. Where the wall is of the cross-section shown in Fig. 16, the proportions shown on the drawing are considered conservative and safe, and while no element of economy exists, the construction is not excessively heavy. The principal dimensions are given in terms of the height, which is represented by 1, or unity. For instance, assume that a retaining wall is 15 feet high and that, according to the figure, the base is .4 times the height, then the width of the base should be 15 × .4 = 6 feet, while the width of the top of the wall should be 15 × .1666 = 2 feet 6 inches. Such a wall will be secure against overturning, sliding, and bulging, with the usual filling, subjected to the most unfavorable conditions of moisture and vibrations.

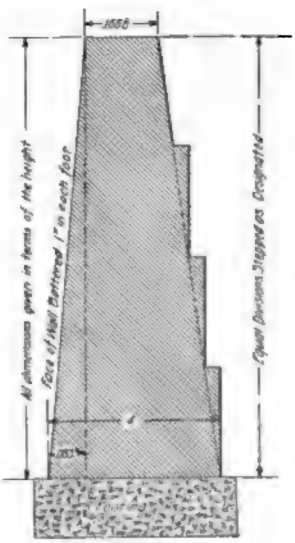


FIG. 16

14. The direct cause of the failure of the greater percentage of retaining walls is due to faulty foundations; consequently, before their construction is attempted the character of the soil should be ascertained, and if doubtful, its bearing capacity determined by testing. The soil is subjected to the greatest pressure at the outside edge of the footing; where it is plastic and will not sustain, without appreciable settlement,

a load of at least 2 tons on each square foot of surface, piles should be driven along the outside edge of the footing course, if the wall is over 12 feet in height. This method of increasing the bearing capacity of the soil underneath the footing adjacent to the point around which the wall

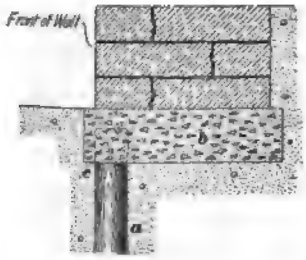


FIG. 17

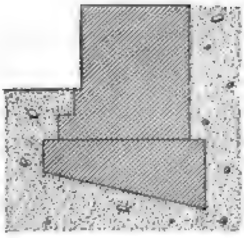
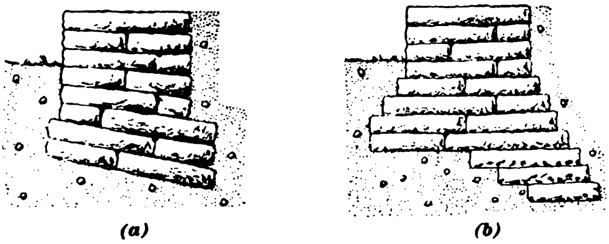


FIG. 18

tends to revolve is shown in Fig. 17. The spruce piles *a* are capped by the concrete footing course *b*. The piles being embedded in the concrete at the top, effectually prevent the wall from sliding along the bearing plane of the soil, besides giving increased bearing resistance to the soil *c* where most needed.

Where there is danger of the wall sliding, the bottom surface of the footing should slope inwards, as shown in



(*a*) (*b*)

FIG. 19

Fig. 18; in such instances, the footings are preferably made of concrete, well proportioned and carefully laid and tamped. If the footings are built of stone and are to have a sloping bed on the soil, the courses may be laid either parallel with the bed or in horizontal planes, as shown in Fig. 19 (*a*)

and (*b*), respectively. Some of the most conservative engineers claim that the method shown in (*a*) is the better, for the reason that when the courses of masonry are inclined in this manner the upper portion of the wall cannot slide on the lower, as it may when the courses are level and the resistance to sliding depends only on the friction and the adhesion of the mortar at the joint. Others think that the method of constructing the footing shown in (*b*) is superior, for any advantage of the inclined bed is more than offset by the fact that the courses in (*b*) are level throughout the wall, which condition facilitates construction and insures greater solidity. The footing courses should be placed well below the frost line, usually at least 3 feet below the surface level.

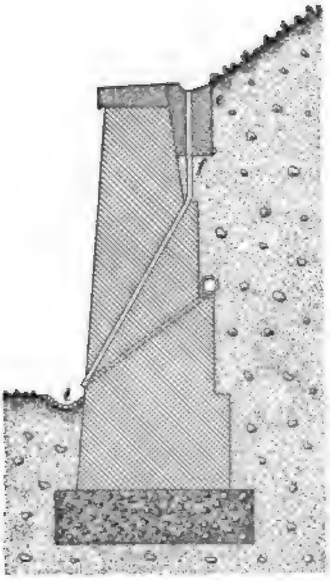


FIG. 20

15. The back of the retaining wall is usually left rough, and if made without a batter the ends of large headers are allowed to project, as they tend to increase the friction between the earth and the backing. The best practice is to step the back of the wall, as shown in Fig. 20. The principal objection to this construction is that surface or spring water is likely to be impeded and to flow into the wall. This objection is overcome by draining the back of the wall as shown in the figure. The face of the wall is usually battered, but a slope greater than 1½ inches per foot is not considered good practice, as there is a liability that the rough surface and joints of the wall will retain rain water, which would cause damage either by freezing and destroying the pointing or by being conducted to the interior of the wall through the joints.

16. The top of the wall should be protected with a coping, preferably one stone in width, designed as shown in Fig. 21. This stone should be cut with a ***wash*** on the top; that is, with the top surface sloping to the outside of the wall as at *a*, so as to shed the rain water. The stone should project beyond the outside of the wall from $1\frac{1}{2}$ to 2 inches and should be provided with a *drip* as at *b*, so that the water shed by the coping stone will drip clear of the top joints of a wall with a batter and entirely clear of a vertical wall. For this reason the retaining wall with a vertical face possesses an advantage over one with a battered face. The back edge of the coping should slope forwards somewhat, or away from the filling, in a line coincident with the sloping back surface of the upper portion of the wall, as shown at *c*. This sloping surface at the top of the wall is necessary so that the upheaval of the earth from frost and the freezing of surface water will not raise the top stones. Sometimes a coating of cement mortar is employed, as shown at *d*, in order to provide a smooth surface for the earth to slide against in rising. The efficiency of this is somewhat doubtful.

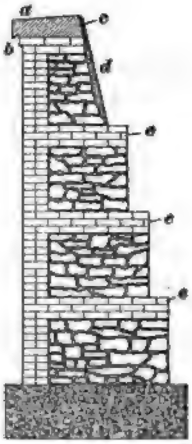


FIG. 21

17. A retaining wall may be built of either squared or rubble masonry laid dry or in mortar, or it may be built of brick, or of brick face and rubble backing, or the reverse, rubble face and brick backing. In all cases the wall must be thoroughly bonded. If of masonry, the bond stones should extend entirely through the wall if possible, and if built with a brick face and a rubble back, the brickwork should be built with header courses, as shown at *e, e*, Fig. 21. On account of the liability of water collecting at the back of a wall and around the footings, the walls should always be laid up with cement mortar composed of one part of Portland cement to two or three parts of sand, depending on the

quality of the work. It is also well to use cement mortar instead of lime mortar, for the reason that the cement mortar attains its *initial set* or first hardening in a few hours, whereas lime mortar remains green several days, thus necessitating the exercise of great care in dumping the filling or backing, as otherwise the wall is likely to be disturbed. Where the wall is laid up in lime mortar, the filling should be dumped in shallow horizontal layers well tamped in place, or the wall should be temporarily supported by shores. Great care must be exercised in all this work to prevent serious accidents.

18. Means are provided for draining the backs of retaining walls by leaving *weep holes*, through the wall, one usually being provided for each 5 or 6 square yards of surface. They are formed by leaving a space 3 or 4 inches wide and the height of a course, between the ends of stones, as shown at *a a*, Fig. 22. Through brick retaining walls, the weepers are usually formed of terra-cotta, lead, or copper pipe. When the filling is surcharged, that is, carried above the level of the top of the wall, as an embankment, it is necessary to provide a cement or tile gutter *e*, Fig. 20, which is drained by the pipe *f* and prevents any surface water from running down the slope of the surcharge and thence down the back of the wall, causing a change in the nature of the filling and probably damaging the masonry by freezing. Such a drain must be provided with a rubbish guard or grating, usually of cast iron, at its upper end. It is well, also, to provide a gutter of concrete, stone, or tamped pebbles at the bottom of the face of the wall *i* so that the water dropping from the weepers or drains will be carried away from the footings. Open drains *b*, *b*, Fig. 22, are

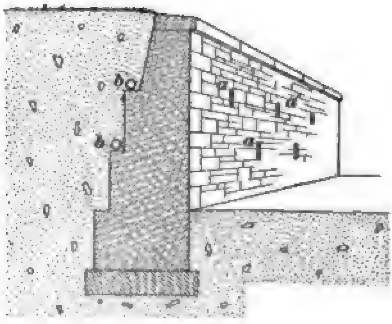


Fig. 22

often provided to conduct the water directly to the weepers or drains, and where the ground is of a clayey nature it is advisable to pack loosely a 6-inch vertical layer of broken stone back of the wall so as to allow the water to drain off more readily than from a retentive filling.

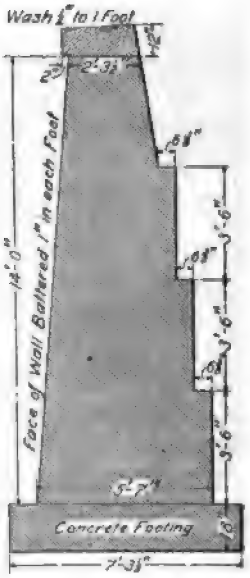


Fig. 23

19. The working section for a retaining wall 14 feet high, of coursed rubble, designed according to the data given, is shown in Fig. 23. The design represents the best practice, and if it errs it has the advantage of being conservative and on the side of safety.

20. Surcharged walls are subjected to a greater pressure than walls where the earth is level with the top, their stability being ascertained most conveniently by the method described in Art. **11.** In applying the principles explained in connection with the stability of retaining walls, it is first necessary to calculate the pressure P by formula **6.** In doing this, however, the weight of the prism of earth whose section is $a\,c\,b\,e$,

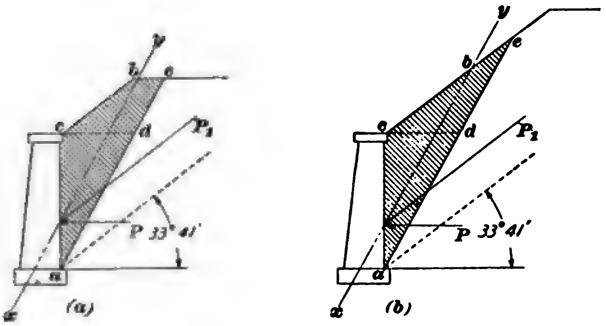


Fig. 24

as in Fig. 24 (a), is considered instead of the triangle $a\,c\,d$, as formerly, while, if the surcharge is so high that the slope of

maximum pressure intersects the slope of the embankment above the wall, as shown in (*b*), the weight of the triangular prism whose section is *a c e* is taken instead of *a c d*.

The direction of the pressure P is considered as perpendicular to the face of the wall and the direction of P_1 is assumed to be at an angle of 33° 41′ with the line of the pressure P. The point of application of the forces P and P_1, however, may not always be located at a distance of one-third the height of the wall from the base in considering a surcharged wall, but it is determined by drawing a line parallel with the slope of maximum pressure and through the center of gravity of the prism of maximum pressure. In Fig. 24 (*a*), *x y* is the line parallel with the slope of maximum pressure *a e* and passing through the center of gravity of the prism *a c b e*; this line intersects the back of the wall at a point somewhat higher than one-third the height, as will be observed from the figure.

When the surcharge is so high that the prism of maximum pressure is triangular, as at *a c e*, Fig. 24 (*b*), the line *x y* will intersect the back of the retaining wall at a distance from *a* equal to one-third the height of the wall, so that the original assumption is not disturbed. In the latter case, there is no additional pressure from the weight of the earth above the point *e* and the surcharge may be as much higher as required without causing additional weight on the retaining wall or necessitating a change in its design.

EXAMPLE.—Determine the stability of the wall shown in Fig. 25, assuming that it is built of granite ashlar.

SOLUTION.—The weight of granite ashlar is 165 lb. per cu. ft. and the weight of dry earth moderately rammed may be considered as 90 lb. per cu. ft. The pressure P, according to formula **6**, is equal to $\frac{W' c d}{h}$, and the pressure P_1 may be determined by the graphical method described in Art. **11** and shown at *i f*, Fig. 14, or it may be determined approximately by the formula $P_1 = .643\, W'$, in which W' equals the weight of the prism of maximum pressure *a c e*, Fig. 25. The cubical contents of the wall, neglecting the footings and considering a portion of the wall 1 ft. in length, is $\frac{2.833 + 2}{2} \times 10 = 24$ cu. ft., and its weight

is $24 \times 165 = 3{,}960$ lb. The cubical contents of the prism of earth $a\,c\,e$, 1 ft. in thickness, is $\frac{11.666}{2} \times 4.75 = 27.71$ cu. ft., and its weight is $27.71 \times 90 = 2{,}494$ lb.

By substituting the weight of the earth, 2,494 lb., for the value of W in the above formula, the pressure P_1 is $2{,}494 \times .643 = 1{,}604$ lb.

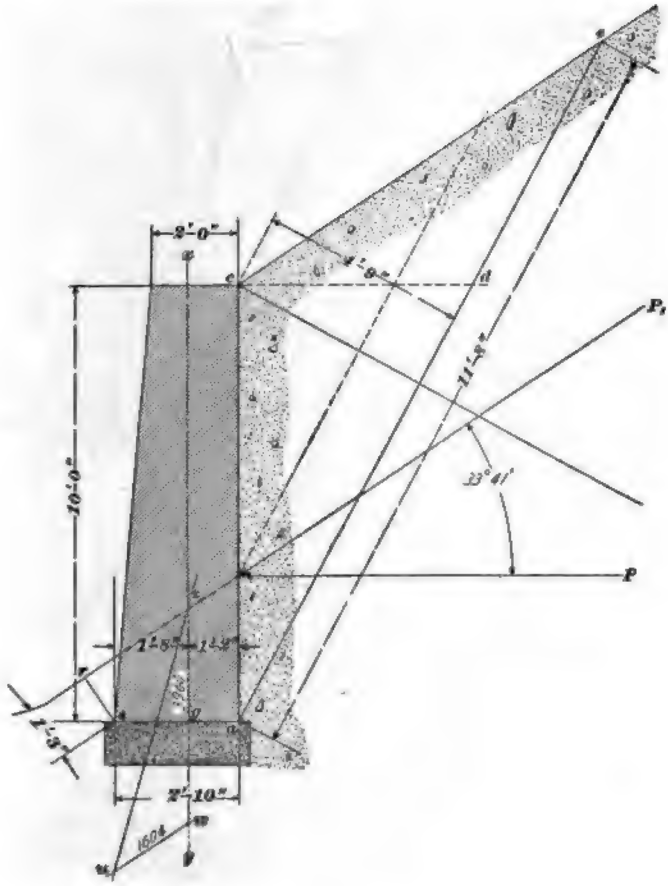


Fig. 25

The stability of this wall may now be determined by the graphical method by laying off, to scale, from z downwards, on the vertical line $x\,y$, which passes through the center of gravity of the wall section, a distance $z\,w$ equal to the weight of the wall. From w, the amount of the pressure P_1 is measured to the same scale, on a line parallel with

P_1 and the point a is thus located. The line $o\,a$ is the resultant of the weight of the wall and the pressure P_1 and cuts the base of the wall at the point t, which is at a distance from the outside edge greater than one-fifth the width of the wall. From this, it is determined that the wall is secure against immediate failure about the outside edge of the footing, provided that the soil is of good bearing capacity and the moment of the weight of the wall about s is equal to or greater than the moment of P_1 about the same point.

The lever arm of P_1 about the point s is 1 ft. 3 in., according to Fig. 25, and the overturning moment is, in consequence, $1{,}604 \times 1.25 = 2{,}005$ ft.-lb. The resisting moment due to the weight of the wall is $3{,}900 \times 1.667 = 6{,}500$ ft.-lb.; it is therefore evident that the wall is well within the safety limit as far as theoretical considerations are concerned. However, the proportions of this wall do not agree with those required by the empirical rules for a wall having much less pressure, as explained in Art. **13**, and it might fail from frost, vibrations, or a change in the nature of the soil due to heavy rains. Ans.

FIREPROOFING

(PART 1)

INTRODUCTION

1. Fireproofing, with its several materials and methods of construction, has reached its present development in the last 20 years; in fact, its introduction and growth have been parallel with that of steel construction. When steel construction was introduced, it was immediately recognized that in order to make steel a safe material to use, and to accomplish one of its principal purposes of adoption, namely, to construct a fireproof building, it was necessary to protect it by some refractory material. Since the steel frame in skeleton construction composes the entire structural fabric of the building, and its destruction would mean the complete collapse of the structure and the endangerment of adjacent buildings, much thought has been given and elaborate tests have been made to perfect adequate fireproofing materials and their employment in efficient systems of protection against fire. The creation of a great demand for a perfect fireproofing system, due to the extensive adoption of steel construction and also to the uncertainty as to the ultimate success of such a system, has led to the invention and the placing on the market of more than fifty types of construction. Many of these systems of fireproofing possess their peculiar advantages and are designed to fulfil the conditions and requirements of modern construction. Structures of different types are subjected to essentially different conditions, and one fireproof system may be used in a certain structure to better

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advantage than another. To choose the best system for th building or structure in hand requires an intimate knowledg of the construction and the materials employed in all systems consequently, the subject of fireproofing becomes a science and an important branch of structural engineering.

PURPOSE AND ADVANTAGES OF FIREPROOFING

2. The term **fireproof** is a fallacy, for the fireproof building, as constructed, does not possess immunity from fire, and there is probably not an absolutely fireproof building in existence. Fireproofing must necessarily extend not only to the building, but to all of the finish and furniture within its walls, and the attainment of a thoroughly fireproof system necessitates the storing of all combustible materials in fireproof vaults, built as veritable furnaces. The interior trim and exterior window casings would need to be of metal, and art metal construction has made such progress that the furniture could be made of the same material and suffer no loss in beauty or utility. Such a building would be fireproof, not because of its construction alone, but from the absence of any combustible materials within or without. A structure, to be proof against a conflagration, including the combustion of all of the goods or merchandise within its walls, would be monumental in character, and as a commercial investment would be practically impossible. The only true fireproof building, then, is one that precludes the possibility of a fire starting; and the best that can be accomplished is to so construct the building that it will hold together until its contents have been consumed.

3. The **advantages** attained by carefully planned systems of fireproofing are several. The spreading of fire is prevented, so that a fire starting in one compartment of a fireproof building, which would threaten the destruction of an ordinary building, does little damage, as it can be confined to the locality in which it started. If the fire does gain headway in a building and burns until the combustible material is all consumed, the fireproofing protects the structural

framework, and prevents its collapse. The insurance rates are considerably lowered, and the saving in this expense will often pay fair interest on the additional cost occasioned by the use of a complete fireproofing system.

It is not the architect and the engineer alone who realize the importance of the extensive use of fireproofing materials in connection with steel-framed structures, but the owner as well sees in it a profitable investment. Guests or tenants in hotels or apartment houses have a feeling of greater security in such a building, and a greater patronage is assured.

GENERAL REQUIREMENTS

4. Conservative practice requires that all buildings of skeleton construction or of the steel-framed type should be fireproof throughout and that every hotel, apartment or tenement house, school, theater, jail, asylum, or institution, should be constructed with walls of brick, stone, Portland cement concrete, iron, or steel, and that no wooden beams or lintels should be placed in the floors or walls. The roofs should be made of a fireproofing material that has been found adequate through a series of severe tests. In such constructions, no woodwork or other inflammable material should be used in any of the partitions or for furring on the ceiling, except when the building does not exceed 150 feet in height; then the doors and frames, together with the interior trim, may be of wood, provided they are filled solidly at the back with incombustible material. Floor boards, and the sleepers to which they are attached, may be used, but the spaces between the sleepers should be solidly filled with concrete, or a similar material that extends to the under side of the floor boards.

In all buildings that are more than twelve stories in height, or exceed 150 feet, the floor construction should be of stone, cement, rock asphalt, tiling, or some similar incombustible material. Wooden sleepers and flooring may be used in the floors, provided the timber is treated with some fireproofing process. All window frames and interior trim may be of

wood, but they should be covered with metal, or so treated as to be rendered fireproof.

5. The important consideration in the design of a fireproof building is to divide the interior into thoroughly fireproof units. This is accomplished by making all hall or permanent partitions between the rooms of a fireproofing material. These should not be placed upon wooden sills nor upon the wooden floor boards, but should be built upon the fireproof-floor construction and extend to the fireproof beam casing above; the tops of all door and window openings should be at least 12 inches below the ceiling.

6. In **fireproof-floor construction,** the steel beams should be so arranged and spaced and their spans so reduced by supporting girders and columns, that the weight of the materials used in the construction will not cause a greater deflection than $\frac{1}{30}$ inch for each foot of span. In fact, conservative practice dictates that the deflection should not exceed this amount for the total load, that is, the weight of the materials of construction and the live load placed on the floors. All steel beams used in fireproof-floor construction should be tied together at intervals of not more than eight times their depth, so that they will be secured against lateral deflection during the construction of the system, and somewhat restrained against twisting and warping in case of fire.

Between the steel beams or walls erected to support a floor system, there should be used a system of fireproofing whose durability and refractory properties have been deter-

to the beam in such a manner as to resist not only the great heat to which it may be subjected, but also any reasonable blow or sudden cooling that might be produced by a stream of water from a fire-hose.

Pipe openings made through fireproof floors for the purposes of plumbing or electrical installation should be filled in with fireproofing materials after the pipes and conduits are in place, thus preventing the passage of fire to other apartments. In no case should a hole larger than 8 inches square be cut for pipes or conduits through any floor system after it is in place and finished.

7. **Interior columns,** whether of cast iron or built up of steel plates and rolled shapes, used to support the fireproof-floor system, should be protected with some fireproof material not less than 2 inches thick and firmly secured in place. All lugs or brackets on these columns that support the floorbeams or other members should be encased, as well as the shaft of the column, though where necessity demands it, they may project within $\frac{7}{8}$ inch of the outer surface of the material.

FIREPROOFING MATERIALS

QUALIFICATIONS AND PROPERTIES

8. **Fireproofing materials** must fulfil many requirements. They must have refractory properties; that is, they must be indestructible by great heat. The construction must be practical so that it can be put in place at a reasonable cost, and with considerable rapidity. The materials must not only resist great heat, but they must be capable of withstanding heavy blows from falling weights and disintegration from sudden cooling. They must be light in weight, for the weight of the fireproofing materials is no small proportion of the entire weight of the structure; consequently, any unnecessary weight in the fireproofing system involves an increase in the sectional area of the columns, beams, and girders throughout the entire building, necessitating a considerable expenditure, from which there is no direct return.

The materials employed in modern fireproofing are not so numerous as the different systems of construction; they consist of refractory materials that will not readily transmit heat and they possess, to a certain extent, all of the qualifications mentioned. They consist principally of charred wood; clay products, such as brick and terra cotta; and monolithic materials reenforced by steel or iron, or supported by a centering composed of metal ribs and wire lathing, terra cotta, or composition forms.

9. **Brick arches** and steel beams were the first elements used in modern fireproof construction. The beams are spaced 4 or 5 feet apart and the brick arches are sprung from their lower flanges with a rise, in inches, usually equal to the span, in feet. The thrust of the arches is taken up by abutting arches, or by tie-rods, spaced every 4 or 5 feet along the length of the steel beams. The haunches of the arches are filled in with concrete, level with the top of the steel beams, and the flooring placed upon sleepers embedded in the concrete.

Brick is also used to encase the columns and in the construction of fireproof vaults, and brick walls are used as fire-stops. In fact, as a fireproofing material, brickwork is excellent, but it has two serious disadvantages: cost and weight. Another possible disadvantage in the use of the brick arch is the fact that on sudden cooling the bricks are apt to crack and break, on account of the internal strains produced.

10. The **terra-cotta tile arch** is the outcome of the efforts to use the refractory properties of brick clay in the production of a light and at the same time strong system of fireproofing and floor construction. Two kinds of terra cotta, known as *dense* and *porous*, are used for this purpose. The **dense tile** is made of selected clay, in practically the same manner as brick, its superiority over brick being due to the hollow form in which it is molded. **Porous tile** is made of clay mixed with sawdust or other combustible material. The compound is thoroughly mixed, molded into the required

shapes, and when sufficiently dry is placed in kilns and subjected to an intense heat, which consumes all the combustible material, leaving the brick or tile in a thoroughly porous condition, and consequently making it much lighter than the dense tile.

Because of their incombustible and non-conductive properties these terra-cotta products are very effective in preventing fire and confining it to one locality when it has

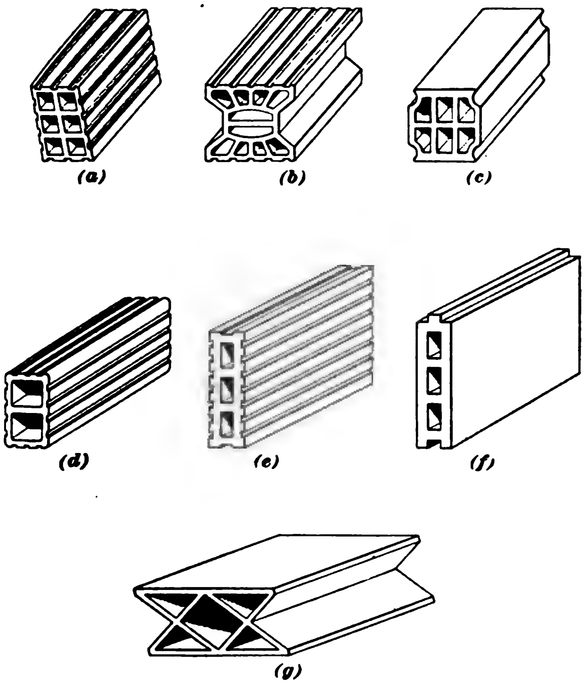


Fig. 1

once started. In Fig. 1 are shown the several forms in which the terra-cotta tile is employed in building construction; (*a*), (*b*), and (*c*) are the usual tiles employed in the floor systems; (*d*), (*e*), and (*f*) are the types used in the erection of fireproof partitions; while (*g*) is the tile often adopted in light floor or fireproof-roof construction. On account of the hollow construction of these tiles and the uniform thickness

of the walls and webs, the clay is uniformly burned and the likelihood of internal strain in the material is small. When used as partitions, the porous tiles possess a peculiar advantage in the fact that they will receive nails readily and hold them with considerable tenacity.

Floor arches carefully constructed of the tile shown at (*a*), (*b*), and (*c*), Fig. 1, have proved strong enough to sustain any load the floor has been required to carry. Both the dense and porous tiles are used in floor systems, but the advantage is with the porous tile, as it possesses more refractory properties, is as strong as the dense tile, and does not transmit heat so readily.

11. **Expanded metal** and **woven wire**, materials that are extensively used in fireproof construction, may be divided into two classes, according to the ends to be attained: first, those having comparatively large meshes and heavy strands, used in concrete floors, roofs, etc., where loads have to be carried; second, those having small meshes, with thin strands, used as lathing, upon which mortar or other plastic material can be applied. When used in floors, the expanded metal is embedded in a slab of concrete placed between the steel beams, the expanded metal supplying the necessary tensile strength to the lower portion of the concrete slab.

When used for fireproof partitions and for protection around columns and beams, the metal lath holds the mortar between its meshes in such a manner that it is securely keyed, and will be able to resist considerable damage from falling materials or from streams of water. In such construction, an air space is usually left between the fireproofing material and the column or beam to be protected.

Expanded metal is made from sheet steel, cut with the grain and pulled, or expanded, into diamond-shaped meshes as shown in Fig. 2. In this way, the area of a sheet is increased about eight times with a corresponding decrease in weight per unit area, and without any waste of material. The steel is tough and fine of texture, and, as shown in the figure, each mesh is independent of the other; numerous

strands may be cut without materially weakening the sheet, and as the expanded metal is cut from a single plate it is impossible to ravel the lath. The thickness of the sheet varies from No. 4 gauge, or about $\frac{1}{4}$ inch, to No. 27 gauge, or .017 inch. The expanded-metal lath used as a means of supporting fireproof plastic material is generally made of No. 27 gauge, while for floor construction Nos. 10 and 16 gauge are usually employed.

The uses of expanded metal are daily increasing with the advancement made in concrete construction, but probably its greatest use at present is as an element of fireproof construction in the erection of floor systems and partitions.

Several fireproofing systems are based on this means of reenforcing the concrete used to fill in the space between the

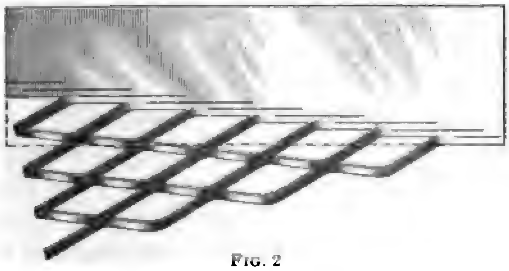


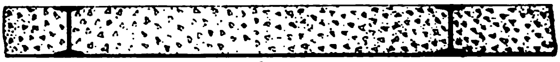
FIG. 2

steel beams, and while expanded metal seems to be best suited for this purpose, other materials employed are galvanized twisted-wire and woven-wire lathing. Woven wire, as used in the principal system of floor construction employing it, is placed in the form of an arch between the steel beams, where it acts as a centering for the concrete filling above. When set, this forms a monolithic or single slab, arched and reenforced on its under side. Both woven wire and expanded metal have unlimited uses in the support of suspended ceilings, and are the means of forming elaborate plastic decorations around beams and columns, answering the double purpose of decoration and fireproofing.

12. Cinder-and-stone concrete, which is used in connection with expanded-metal and woven-wire lath, is also

employed in floor systems, when it is designated as *monolithic construction*. In such systems, the concrete is tamped in place between the beams on temporary centerings placed beneath. The floorbeams may be of special rolled shapes, designed so as to obtain the greatest possible transverse strength with

(a)



(b)

FIG. 3

the least weight, as in Fig. 3 (*a*), or they may be light steel I beams thoroughly encased in the concrete, as at (*b*). Sometimes wooden beams, reenforced longitudinally with steel angles bolted or secured by lagscrews to the sides of the beams, as in Fig. 4, are used. These angles are often cambered or sprung, while cold, into the form of an arch upon the sides of the wooden beams, and are held in this shape by bolts through both the angles and the beam. When the concrete sets and the angles are thoroughly embedded they reenforce the concrete, and though the wooden beams act as nailing strips, they afford little transverse strength to the floor system and can rot away without materially impairing its strength.

In all of these systems the beams supporting the concrete slabs are usually spaced from 18 to 24 inches on centers, and on account of this short span the concrete slab between the beams requires no reenforcement, even when not more than

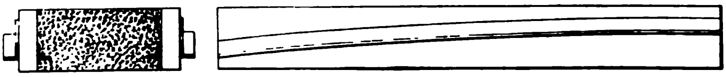


FIG. 4

4 or 6 inches deep, and is amply strong for the usual floor loads. The strength of the beams is materially increased by being embedded in the concrete, for they are held rigidly against lateral deflection. If the beams are so designed as to offer some holding power or adhesion to the concrete and the

concrete extends above their upper fibers, the combination of a concrete beam reenforced by the steel angles, which act as a tension member, is obtained and a greater strength is realized than would be from either the concrete or steel beam alone.

13. Calcined gypsum, or **plaster of Paris,** is used in several fireproof-floor systems and also to protect columns and girders, where its adaptability to decorative work makes it especially desirable. This material is a non-conductor of heat and possesses the advantage that it may be poured into molds or worked into any shape. It may be used as concrete, in slabs, supported between beams and reenforced by steel wires or rods, or it may be molded into lintels that are supported upon skew backs between the floorbeams. The lintels effectually fill the space when placed side by side and possess the required strength when reenforced by woven

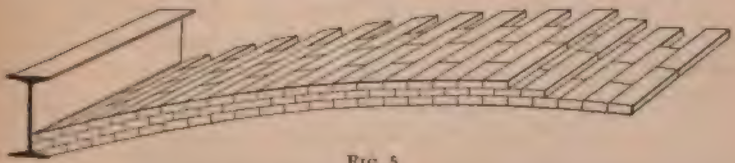


FIG. 5

wire or expanded metal. One system uses a segmental tile arch, which rests upon the lower flanges of the I beams supporting the floor and forms a permanent centering upon which concrete may be tamped; a strong floor system and finished ceiling are thus secured. In the construction of one floor system, wood shavings are introduced in the plaster of Paris with a view of lightening the composition and giving greater tenacity to the materials.

14. Spanish tile arches are used in fireproof-floor, roof, and dome construction. They are peculiarly light, economical, and scientific in their design and form the *Gustavino construction*, which is named from its inventor, a Spanish architect. It consists essentially of three layers of tile, 1 inch thick, that are thoroughly embedded in cement and are laid up in the form of a segmental arch, as shown in Fig. 5.

The efficiency of this construction lies in the fact that the segmental arch in which the tiles are laid approximates closely the line of pressure in the arch, and consequently there exists little tension in the voussoirs or tiles. If the line of pressure does fall without the arch and a tensile stress is created, it is resisted by the adhesion of the Portland cement in which the tiles are laid, and by the shearing strength of the tile. The strength of these arches is surprising when the thickness, 3 inches, is considered; they have been found to sustain, on reasonable spans, a load of 700 pounds per square foot. The arch has been employed for spans as great as 20 feet, while domes 48 feet in diameter and provided with no other support have been built by this method. The construction is especially adaptable to large spans between heavy masonry or brick walls that, when sufficiently heavy, resist the thrust of the arch, and no tie-rods are required. However, when the walls are light and the construction is placed between steel beams, the thrust must be provided for by introducing tie-rods.

The appearance of this construction, from beneath, is pleasing, as the under side of the first layer of tiles may be glazed, and where no tie-rods are used the vaulted effect is architecturally suited to such purposes as the flooring or covering of pavilions, cloisters, or galleries, and passages in public buildings, churches, or libraries.

FLOOR CONSTRUCTION

TESTS

15. Tests of the various systems of fireproof-floor construction are required by the building departments in many of the larger cities, before they are accepted as safe construction. The laws regulating the methods by which these tests are conducted vary somewhat in detail, but the

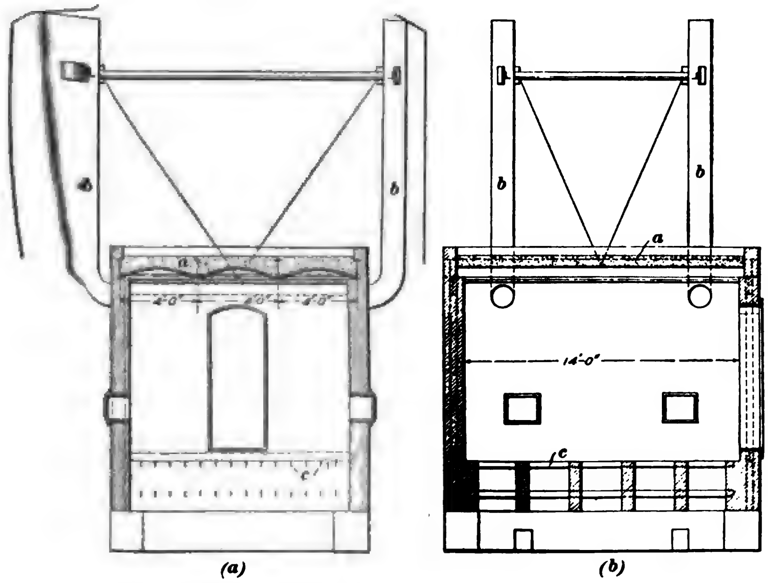


FIG. 6

conditions to be fulfilled are essentially the same. The New York building laws require that such tests shall be made by constructing within enclosing walls a platform *a*, as shown in Fig. 6, in which (*a*) represents a tranverse and (*b*) a longitudinal section. The platform co[illegible] of four rolled-steel

beams, 10 inches in depth and weighing 25 pounds per lineal foot, which are placed 4 feet between centers and are connected by transverse tie-rods. The walls of the enclosure are so constructed that of the four beams the two interior ones have a span of 14 feet; the two outer beams are supported on the side walls throughout their length. The fireproof construction to be tested is filled in between the beams, care being exercised to protect the exposed parts of the beams. The construction of the fireproof system for the test must be, in all cases, as employed in actual practice, and no more material is allowed than would ordinarily be used in the system. The ceiling is to be plastered below, as in the finished work, and the floor construction loaded with a uniformly distributed load of 150 pounds per square foot, the entire weight being carried by the filling. A wood fire is built in the enclosure on the grate *c*, the four flues *b* being provided for the escape of the smoke, and an average temperature of 1,700° F. is maintained for not less than 4 hours, during which time the test floor, or platform, must remain in such condition that no flame will pass through any part of it and no part of the load fall through. A stream of water, under a pressure of 60 pounds, is directed for 5 minutes against the bottom of the platform through a $1\frac{1}{8}$-inch nozzle; the top of the platform is then flooded with water under a low pressure, and the stream of water again applied at a pressure of 60 pounds to the bottom of the platform for another period of 5 minutes. In addition to this, a load of 600 pounds per square foot uniformly distributed over the middle bay is applied. The steel beams supporting the floor system shall be afforded such protection from the heat that after this load is removed the maximum deflection of the interior beams on cooling will not exceed $2\frac{1}{2}$ inches.

Such a system of tests applied to any fireproof-floor construction must show its carrying capacity under a static load and its capability of resisting high temperatures with its retention of strength; also, its indestructibility from blows or sudden cooling, or streams of water under heavy pressure. A construction that withstands such a test makes an excellent fireproof-floor system, as it is understood in modern work.

BRICK ARCH

16. The **brick arch** is still used in fireproof-floor construction, but is fast being replaced by patented systems that offer more protection to the steel beams and are much lighter in weight. The construction usually employed is shown in Fig. 7; it consists of a brick arch, commonly $4\frac{1}{2}$ inches, or the width of a brick, in thickness and having a rise of from 3 to 5 inches, springing from the lower flange of the steel beams. Where the floor is extensive in area, the lighter steel beams *b* are secured, at intervals of 4 or 5 feet, to a main girder *a* by bolts or rivets; and in order to prevent the steel beams from spreading, tie-rods *c* are placed through

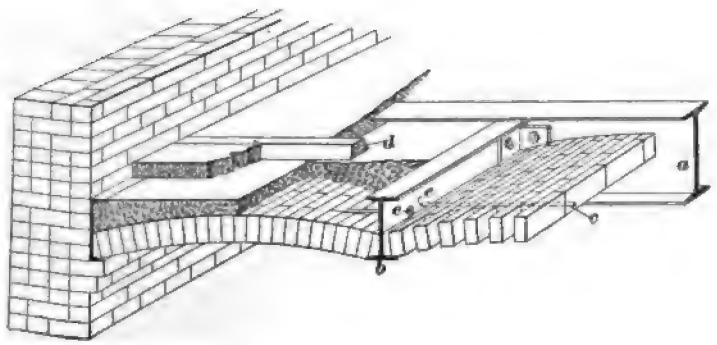


Fig. 7

their webs. These tie-rods are considered necessary in order to take the thrust of the arches, though where the arches are in a series and the end arch abuts a heavy wall, as shown in the figure, they are unnecessary for this purpose. However, they have a tendency to hold the beams in position during construction and prevent their lateral deflection when subjected to heat.

The brick arch is constructed of ordinary hard-burned brick, having a true form and laid so that all end joints are broken, as designated in the figure. The space between the crown of the arch and the top flange of the floorbeam is filled in with concrete, made of either stone or cinders and the best

Portland cement. The flooring is secured to sleepers *d* embedded in the concrete, which should be carried to the under side of the flooring in order that no space will exist between the top of the concrete and the floor, which would form a flue and aid in spreading the fire.

Brick arches are usually constructed upon a wooden center, as shown in Fig. 8 (*a*). The pieces of timber marked *a*, *a* are about 4 inches by 6 inches or 4 inches by 8 inches, and have the upper edges cut to the curve of the intrados of the arch; their length is equal to the distance between the lower

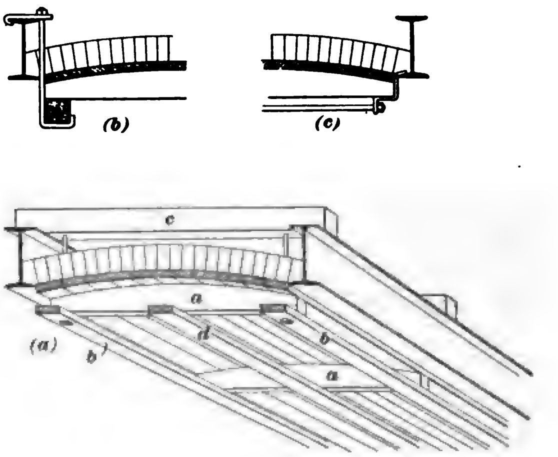


FIG. 8

flanges of the I beams. They are supported on the planks *b*, *b*, which are secured by bolts to the plank *c*, resting on the top of the I beams. The central plank *d* is spiked to the pieces *a*, *a*, to hold them rigid and prevent any lateral deflection. Narrow strips of boarding or flooring are laid on top of the curved planks, as shown in the figure. At (*b*) and (*c*) are shown other methods of supporting the center, which may be employed where more convenient. On the center, a section of the arch is constructed, and when it has set, the centering for this bay is carefully removed and transferred to another section of the floor. The bricks are not laid in mortar, but are placed close together and keyed with

pieces of slate, after which they are thoroughly grouted with cement mortar. The concrete filling may be composed of one part of cement, three parts of sand, and five parts of broken stone, or, if it is desired to have a light filling, furnace slag or ashes may be substituted for broken stone. In using the cinder concrete, a proportion of one part of cement, two parts of sand, and nine parts of cinders has been found to give good results.

17. The **tie-rods** used in this construction are proportioned to withstand the thrust of the arch when it is isolated or not one of a system. The arch, however, is usually one of a system and the thrust is counteracted by the thrust of the arches adjacent; therefore, the conservative practice is to use $\frac{7}{8}$-inch tie-rods from 6 to 8 feet apart, or about eight times the depth of the steel beams. In designing this system of floor construction, it is advisable to place the tie-rods above the intrados of the arch, so that they will not be exposed to view or subjected to heat in case of fire. It will be observed from Fig. 7, that in punching the holes in the steel beams to receive the tie-rods, a hole is required for each tie-rod; that is, there are two $\frac{15}{16}$-inch holes punched 2 inches apart in the web of the central beam, as shown at *c*, *c*.

18. The floor surface may be of finished flooring, tongued and grooved, and laid close upon rough flooring, or of asphalt, cement, or tile. The New York building laws, which have been recently revised and may be considered as the model for the building laws in other cities, present the following stipulations:

"Brick arches springing from the lower flanges of steel floorbeams shall be designed with a rise such as will carry safely the imposed load, though it shall never be less than $1\frac{1}{4}$ inches for each foot of span between the beams. The brick arch shall have a thickness of not less than 4 inches for spans of 5 feet or less and 8 inches for spans over 5 feet, or such thickness as may be required by the Board of Buildings. The brick arches shall be composed of good, hard-burned brick or of hollow brick of ordinary dimensions laid

to line on centers. The arch shall be properly and solidly bonded, each longitudinal line of brick breaking joints with the adjoining lines in the same ring and with the ring under it when more than a 4-inch arch is used. The brick shall be well wet and the joints filled in solid with cement mortar. The arches shall be well grouted and properly keyed."

The brick-arch type of floor construction is durable, but is comparatively heavy, and the fact that the lower flange of the I beam is unprotected, as will be seen from Fig. 7, offers considerable objection to it; the lower flange of the beam, if subjected to great heat, will loose much of its strength and the destruction of the arch is likely to ensue from excessive deflection. This system is admirably adapted for any location where it is desirable to obtain a strong

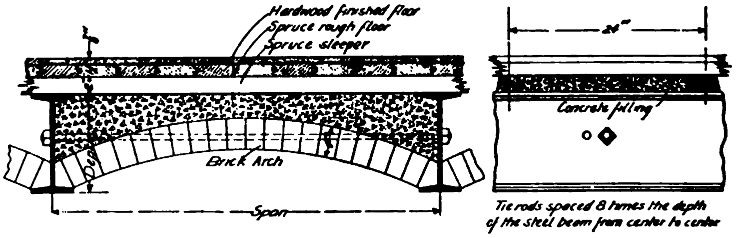


FIG. 9

system of flooring over a cellar or vault that may be shut off from the rest of the building and where the fire will be above the construction. It is used considerably in mill construction in connection with the slow-burning system, where a section of the building is designed to be fireproof, that is, free from the possibility of ignition, though its efficiency in case of an intense fire from a large amount of combustible merchandise is indeed doubtful.

19. The weight of this system of flooring with both stone and cinder concrete filling, for several spans and the usual sizes of steel beams, is given in Table I; the weights are based on a typical section shown in Fig. 9.

In making the calculations for the weights given in Table I, the weight of the brickwork laid in cement mortar was taken

at 130 pounds per cubic foot, while the weight of the stone and cinder concrete was figured at 140 pounds and 80 pounds per cubic foot, respectively. The filling between the wooden sleepers was considered, in each case, to be of the same material as the filling over the arch.

TABLE I

WEIGHT OF FIREPROOF BRICK-ARCH CONSTRUCTION

Span of Arch Inches	Size of Steel Beam		Weight of Brick Arch. Pounds per Square Foot of Floor Surface	Weight of Cinder Concrete. Pounds per Square Foot of Floor Surface	Weight of Stone Concrete. Pounds per Square Foot of Floor Surface	Weight of Steel Beams. Pounds per Square Foot of Floor Surface	Weight of Floor Construction. Pounds per Square Foot of Floor Surface	Total Weight if Stone Concrete Is Used. Pounds per Square Foot of Floor Surface	Total Weight if Cinder Concrete Is Used. Pounds per Square Foot of Floor Surface
	Depth Inches	Weight Pounds per Foot							
36	10	25	49	35.3	63.8	8.3	7	128.1	99.6
36	12	31½	53	38.4	69.5	10.5	7	140.0	108.9
42	10	30	48	35.0	63.	8.6	7	126.6	98.6
42	12	31½	51	39.5	71.4	9.	7	138.4	106.5
42	12	40	51	39.5	71.4	11.4	7	140.8	108.9
42	15	42	56	43.6	79.1	12.	7	154.1	118.6
48	12	31½	49	36.7	66.4	7.79	7	130.2	100.5
48	12	40	49	36.7	66.4	10.	7	132.4	102.7
48	15	42	53	42.0	78.5	10.5	7	149.0	112.5
54	12	40	49	38.8	72.2	8.9	7	137.1	103.7
54	15	42	52	41.3	74.5	9.3	7	142.8	109.6
54	15	50	52	41.3	74.5	11.1	7	144.6	111.4
60	15	42	51	42.7	77.2	8.4	7	143.6	109.1
60	15	50	51	42.7	77.2	10.	7	145.2	110.7
60	15	60	51	42.7	77.2	12.	7	147.2	112.7

20. The strength of the brick arches used in floor construction is considerable, and when properly proportioned and well built they are practically indestructible from any usual load or damage that might occur in a building. In New York a test of a brick arch, constructed as described in Art. **16**, was made by dropping on the top of it from a height of 3 feet, a piece of granite about 15 inches square, rounded on the edges, and weighing approximately

800 pounds. It was only after repeated blows that bricks were dislodged and the destruction of the arch accomplished by the loosing and tearing away of the brick. Other arches tested at the same time were destroyed at the first blow. Tests were made at the United States arsenal at Watertown on brick arches having a rise of $8\frac{1}{2}$ inches, placed between steel beams 15 inches in depth, weighing 65 pounds per foot, and spaced 7 feet 5 inches apart; the bricks were laid on edge in lime mortar and well backed with concrete and planked over. The failure of the system was caused by the failure of the beams, which had a span of 28 feet 6 inches, and did not occur until a weight of 563 pounds was supported by each square foot of floor area. The deflection at the center of the middle beam was slightly over 13 inches, which shows that the arches possessed considerable elasticity.

TERRA-COTTA TILE FLOOR SYSTEMS

21. The **tile-arch system** of floor construction has been extensively used in the erection of fireproof buildings. It consists of hollow clay tiles so arranged as to practically form a flat arch between the steel beams supporting the system. The tile may be either of hard-burned clay or of porous terra cotta.

22. There are two principal methods of construction employed in connection with hollow tile, known as *side* and *end construction*. The **side method**, which was the original construction, is shown in Fig. 10. In it the air spaces in the hollow blocks run parallel with the steel floorbeams. The blocks marked *a*, *a*, which are the *skew backs* of the arch, are so arranged as to protect the lower flange *d* of the steel beam, and to take the thrust of the arch tiles. The central block *b* in the arch is the key tile and acts as a wedge, holding the voussoirs *c*, *c* in position when the centering is removed. This tile takes the place of the keystone in an arch. The skew backs and the key tile can be used only in one position, so they are molded with dovetail grooves, as at *e*, *e*, for plastering only on the bottom surface, while the common tile or

voussoirs are grooved on both the upper and lower surfaces, as they are interchangeable and may be laid either way.

It will be observed that the two central ribs in the skew back, marked *f*, slope toward the lower flange of the **I** beam and that the top edge is chamfered off adjacent to the beam.

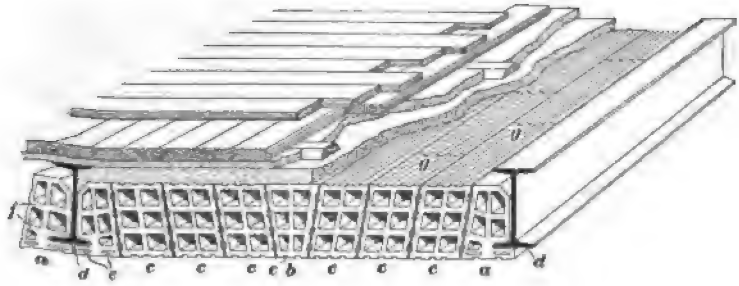


FIG. 10

In this way the final thrust of the arch is directed toward what would be the springing line, and the stability of the arch is increased by forcing the line of resistance well within this middle third and away from the outer edges. All tendency for tension at the outside of the joints on the under side of the arch, which would cause a crushing of the tile and the destruction of the system, is thus eliminated.

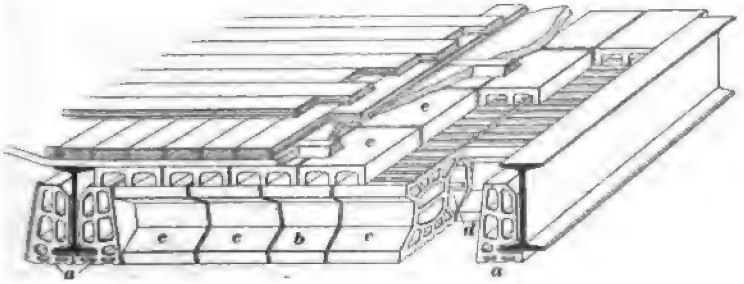


FIG. 11

23. The end method, which is the improved system of flat-tile arch construction, is shown in Fig. 11. The air spaces in the hollow tile extend perpendicular to the steel floorbeams, except in the skew backs, where they run parallel with the beams. The skew backs *a*, *a* are practically the same

as those employed in the side-construction method, but the voussoirs *c* and key block *b* are considerably different, as they take a bearing on the end section and not on the side, and are of such a form that hollow spaces are provided between the tile in a direction perpendicular to the beam, as shown at *d*. These spaces, besides diminishing the weight of the system considerably, provide a means of passing the tie-rods through the arch from beam to beam.

This system is considered superior to the side method of construction because, for a given weight, it is about 25 per cent. stronger and is, consequently, a lighter system of flooring. The openings between the tiles readily permit the insertion of the tie-rods and allow any variation in their

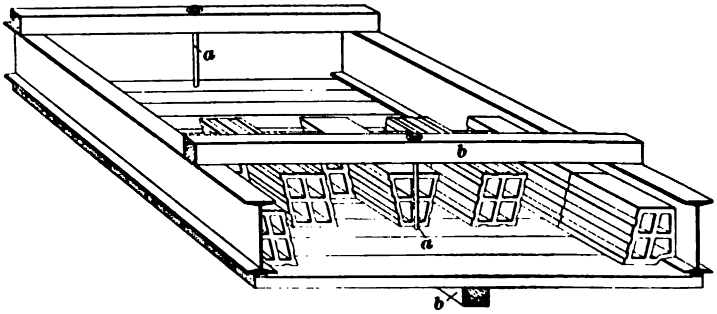


FIG. 12

spacing; thus, the cutting of the tile is avoided and any risk of breakage likely to occur with the side method of construction is eliminated.

24. Both systems of flat-arch construction are erected in the same manner and require a temporary wooden centering. The terra-cotta blocks are laid in place with cement mortar, and in the side method of construction the end joints of the tiles are broken in each adjacent course, as shown at *g*, *g*, Fig. 10. The number of tiles required to form the arch varies according to the size of the blocks and the distance between the steel beams. In setting the arches, a flat centering made of 2-inch planking should be employed. This centering is supported upon 4″ × 6″ joists suspended from

the steel beams by means of clamps secured to the bottom flange. The joists are cut to a curve of a large radius on the top edge so that the centering will have a slight camber, not more than $\frac{3}{8}$ inch for spans of the usual length. After the arch is formed and the mortar has set, the centers are removed and used elsewhere.

Another form of centering that is often employed is shown in Fig. 12. The planks supporting the tile arch during erection are so thin that they may be drawn to a slight camber by the bolts at *a, a* and the straining timbers *b, b*.

The terra-cotta blocks are usually made the same depth as the steel beams, and as they extend about $1\frac{1}{2}$ inches below the lower flange of the beams, the upper surface of the flat arch is about the same distance below the top flange. The space between the top of the tile arch and the finished floor is filled with a poor concrete, as shown in Fig. 10, or with a special hollow-tile filler *e, e*, as in Fig. 11. This filler block is made in various thicknesses, to suit all requirements, and may be used to advantage where the beams are of a considerably greater depth than the arch blocks.

25. The span and depth of the arch determine its weight per square foot of floor surface, which ranges from 27 to 44 pounds for the side method of construction and from 27 to 38 pounds for the end method.

Table II gives the weight per square foot of floor surface for flat arches of different spans in both constructions. The typical sections of both the end and side construction considered are shown in Fig. 13 (*a*) and (*b*), respectively, each illustration consisting of a transverse and longitudinal section. It will be observed that concrete is used as a filler, instead of the tile as shown at *e* in Fig. 11. The meaning of the letters *d* and *m* is seen from the illustrations.

26. The strength of flat-tile arches has been repeatedly tested; an arch constructed by the end method with a span of 10 feet and a depth of 10 inches, sustained a load of sand 1,000 pounds per square foot. Table III gives approximately the safe allowable load for arches of this construction.

TABLE II

WEIGHT OF FLAT TERRA-COTTA TILE ARCHES, EXCLUSIVE OF PARTITIONS

Type of Arch	Depth of I Beam d Inches	Thickness of Arch m Inches	Thickness of Floor $d+6$ Inches	Weight of Arches Pounds per Square Foot	Weight of Filling Pounds per Square Foot	Weight of Flooring Pounds per Square Foot	Weight of Ceiling Pounds per Square Foot	Weight of Steel Pounds per Square Foot	Total Weight of Arch Pounds per Square Foot
Hollow-Brick Flat Arch. Ordinary Type	8	6	14	29	29	7	4	7	76
	8	8	14	35	17	7	4	7	70
	9	6	15	29	35	7	4	7	82
	9	9	15	37	17	7	4	7	72
	10	8	16	35	29	7	4	8	83
	10	10	16	41	17	7	4	8	77
	12	8	18	35	41	7	4	8	95
	12	12	18	48	17	7	4	8	84
	15	8	21	35	59	7	4	10	115
	15	12	21	48	35	7	4	10	104
Hollow-Brick Flat Arch. End-Construction Type	8	8	14	30	17	7	4	7	65
	9	8	15	30	23	7	4	7	71
	9	9	15	32	17	7	4	7	67
	10	8	16	30	29	7	4	8	78
	10	10	16	34	17	7	4	8	70
	12	8	18	30	41	7	4	8	90
	12	12	18	37	17	7	4	8	73
	15	8	21	30	59	7	4	10	110
	15	12	21	37	35	7	4	10	93

In Table III, the approximate safe loads of flat arches, as ordinarily constructed, have been deduced from recent experiments, and factors of safety ranging from 6 to 8 have been

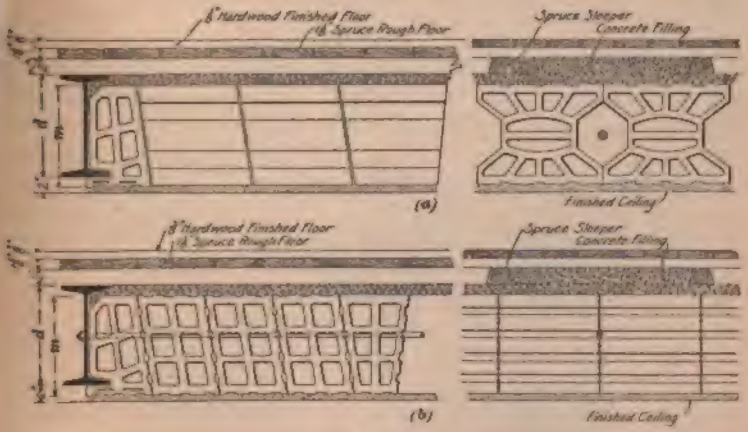


FIG. 13

allowed. Since the arches are often carelessly set, the margin of safety should be greater than would otherwise be necessary.

TABLE III

APPROXIMATE SAFE LOAD ON FLAT-TILE ARCHES, IN POUNDS PER SQUARE FOOT

Depth of Arch Inches	Distance Between Beams				
	4 Feet	5 Feet	6 Feet	7 Feet	8 Feet
6	150	100			
7	200	150			
8	275	175	125		
9	300	200	140		
10	325	225	150	100	
12	400	250	200	125	100

27. A method of protecting girders by means of hollow terra-cotta blocks is shown in Fig. 14; (*a*) shows the

protection for a single girder and (*b*) for a girder composed of two I beams. This protection may be adapted to a girder of any width by changing the width of the piece *a*. Gauged mortar is used for holding these tiles in position.

28. The **Herculean flat arch**, shown in Fig. 15, is an improved type of end construction. In it no skew backs are required, as the arch rests upon the supporting walls or upon the tops of steel beams placed transversely; the joints in alternate courses are broken, and the top of the construction presents a smooth and unbroken surface. The T irons *a*, *a* are thoroughly embedded in Portland cement, and, being encased in a terra-cotta covering that is in no place less than 2 inches thick, they are removed from all possible contact with fire. The arch is generally used in spans from 18 to 20 feet, though it has been used in spans of 22 feet 6 inches. When it is laid upon the top of the steel beams, the system offers entire fireproof protection from above, but the steel beams are exposed beneath. This arch has been approved by the building departments in several of the principal cities and by the United States government.


(a)

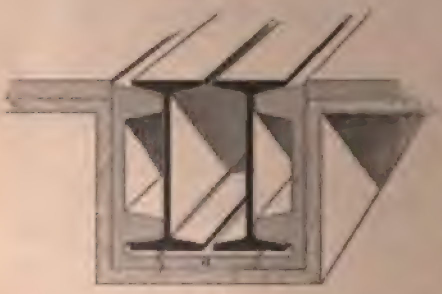

(b)

FIG. 14

The depth of the blocks varies from 8 to 12 inches and they are commonly about 12 inches wide and 12 inches long.

The grooves in which the **T** irons are encased are molded in the blocks and are large enough to accommodate a $1\frac{1}{2}'' \times 1\frac{1}{2}'' \times \frac{5}{16}''$ standard **T**, this being the size commonly employed.

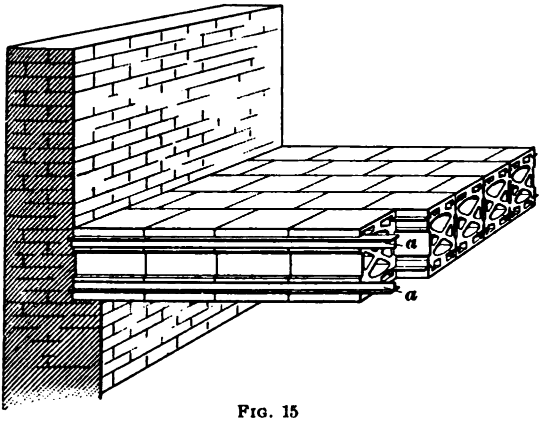


FIG. 15

In constructing this arch, the blocks are embedded in cement and a bearing of from 4 to 6 inches should be allowed on the walls or girders.

The Herculean arch is well adapted to long spans where the tile arch can take a bearing at each end upon brick walls, and though it possesses considerable strength, it is better adapted for light floor construction, such as would prevail in office buildings, libraries, hospitals, and dwellings, rather than in storehouses or warehouses.

TABLE IV

WEIGHT OF HERCULEAN ARCH

Depth of Arch Inches	Weight per Square Foot Pounds
8	33
10	42
12	51

29. The weight of this arch, including **T** irons, is given in Table IV.

30. The **Keystone tile arch,** which forms an extremely light fireproof-floor construction, is shown in Fig. 16. It

consists of skew backs *a*, *a* protecting the steel beams and supporting a hollow slab *b* of terra cotta, the slab, which is made in one piece, being substituted for the flat arch. The tie-rods holding the beams together are passed through openings between the abutting edges of the tile. No centering is required in setting this floor system, and, in consequence,

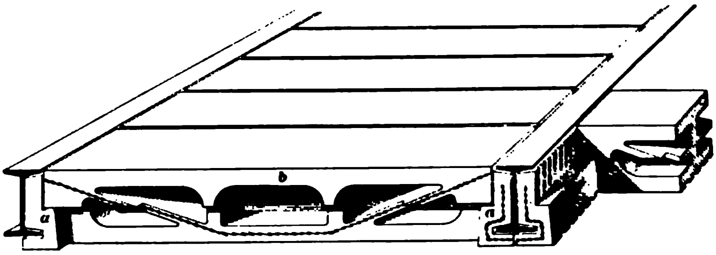


FIG. 16

the erection is greatly facilitated. It is strong, durable, and extremely light in weight. The requirement of the system is that the steel beams shall be spaced 30 inches apart, and they are usually from 5 to 6 inches in depth. For the construction of floors in offices and dwellings, this system is found to be economical.

EXPANDED-METAL, OR MONOLITHIC, CONSTRUCTION

31. The **expanded-metal, or monolithic, construction** is the adoption of sheet steel in connection with concrete for fireproof-floor, partition, and wall construction. The steel, which is employed in the form known as **expanded metal**, reenforces the concrete when subjected to transverse stress and provides a lath, or plastering surface, in partition and wall construction. A slab of concrete has little transverse strength, owing to the fact that its tensile strength is small, and, therefore, by embedding some material adjacent to its lower surface a considerable gain in strength is obtained. Expanded metal has been found to admirably fulfil the requirements and furnish the necessary strength. The concrete slab, being thus reenforced, is capable of

sustaining considerable weight and is adapted for safe fireproof-floor construction. The introduction of steel in connection with the concrete allows the use of thin slabs, and consequently promotes economical construction. The monolithic slab, which constitutes the principal feature of this construction, is practically a beam supported between the

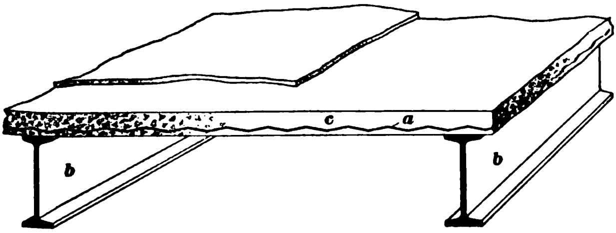


FIG. 17

usual steel beams or girders. The expanded metal is placed close to the bottom of the slab and is proportioned to suit the requirements of load and span.

Fig. 17 shows the simplest form of construction, which consists of the sheet of expanded metal *a*, supported upon two steel beams *b*, *b*, and thoroughly embedded in the concrete slab *c*. This system is adaptable for flooring that is

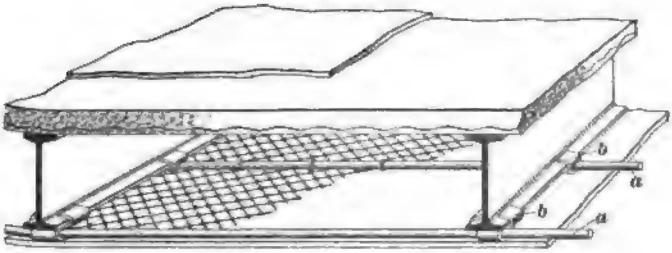


FIG. 18

intended to be fireproof from above, but on account of the steel beams being entirely exposed, it cannot be considered a thoroughly fireproof construction. The exposed beams may be partially protected from below by the use of a suspended ceiling, as shown in Fig. 18. This ceiling is plastered upon expanded metal, which is wired to $\frac{3}{4}'' \times \frac{5}{16}''$

steel channels or $\frac{5}{8}$-inch round iron rods *a*, *a* placed from 12 to 16 inches on centers, which, in turn, are secured to the I beams with clips *b*, *b*. This construction provides a finished ceiling and a protection for the lower flange of the I beams as long as it remains in position.

32. A thoroughly fireproof construction is one in which the steel beams are entirely protected from contact with the heat, as shown in Fig. 19. In the construction shown at *a*, the space between the two flanges of the beam is filled with

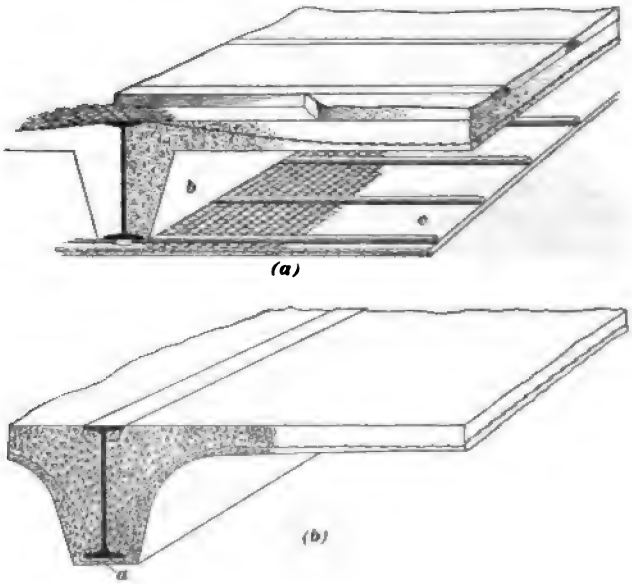


FIG. 19

concrete *b* and the usual suspended ceiling *c* is constructed, so that, even though the ceiling is damaged, the steel beams will still be protected.

Where a paneled-ceiling effect is desired, the type shown at (*b*) may be employed. This construction offers the best protection, for both the bottom flanges and the webs of the steel beams are fireproofed. The steel beam in this construction is protected by enclosing it in concrete and plaster, which is held in place at *a* by the expanded metal bent to

pe around the flange of the steel beam. These systems construction transfer all of the load to the lower flange of steel beams and are admirably adapted where economy height is an object, since there is little loss of headroom,

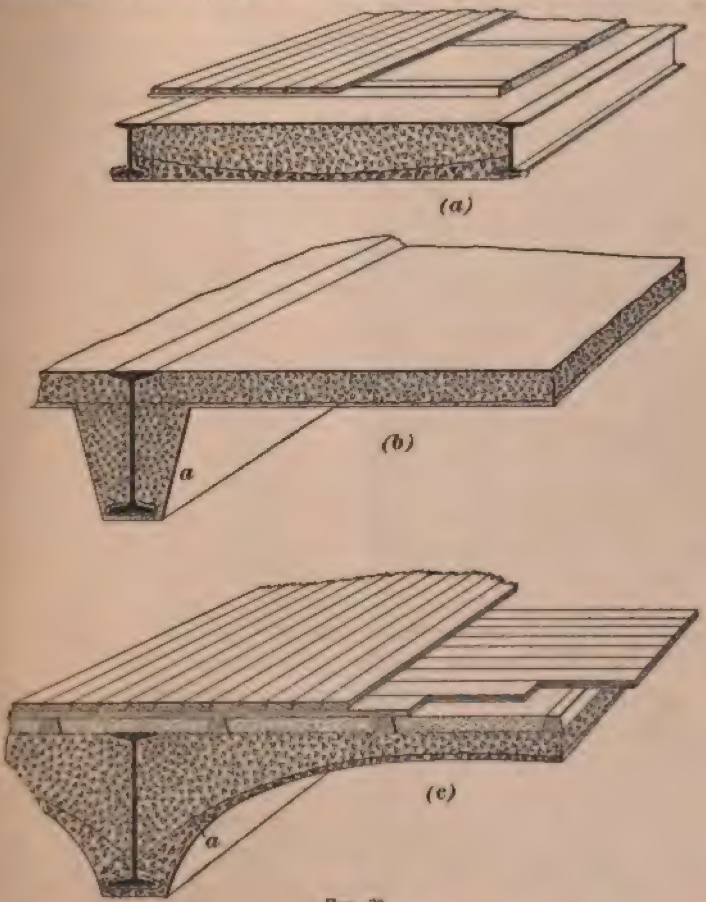


FIG. 20

the construction being entirely included between the flanges of the steel floorbeams.

The adaptability of monolithic construction to the various conditions met with in building construction may be observed from Fig. 20. At (*a*) is shown a construction suitable for

shallow beams from 6 to 8 inches in depth. It off
advantages of concrete fireproofing for the beams an
ceiling with no loss of headroom between the fl
ceiling. It could best be used in an apartment ho
any light floor construction and is most economic
the beams are not over 7 inches deep nor spaced mo
4 feet 6 inches apart. The system shown at (*b*) is e
where beams of great depth are used; it is a devel
from the one shown in Fig. 19 (*a*), and, as will be o
is practically the same except that the expanded meta
flat in the concrete slab and is below the upper flang
slab is incorporated with the concrete filling at *a* a
on it. For heavy work, the system shown at (*c*) i
sively used and combines strength with complete pr
The expanded metal *a* is added to the concrete arch
increase its strength and thus allow a minimum thic
concrete to be employed. The metal also reenf
concrete arch so that any injury to the floor from
or shocks is impossible. In this system of constru
beams are spaced from 4 to 8 feet apart on centers.
the concrete segmental arch, cinder concrete is
filling, in order to bring the construction flush wit
of the beams.

33. Besides the several systems of floor co
using expanded metal as a means of reenforceme
are two special types of construction that employ p
the same materials but introduce an additional st
member of metal. The systems are known as th
and *Suspension* types of floor construction, and diff
the first reenforces the monolithic floor slab by me
arch, while the second accomplishes the same end b
or bar of iron supported between the upper flange
steel floorbeams.

34. The **Golding system** of fireproof-floor co
is shown in Fig. 21 and consists of the usual con
strengthened by the steel channels *a*, *a* sprung bet
lower flanges of the beams and having the span

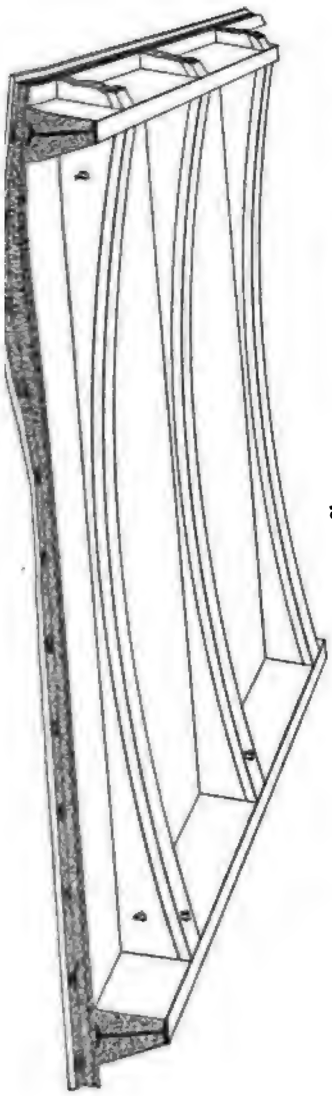


FIG. 21

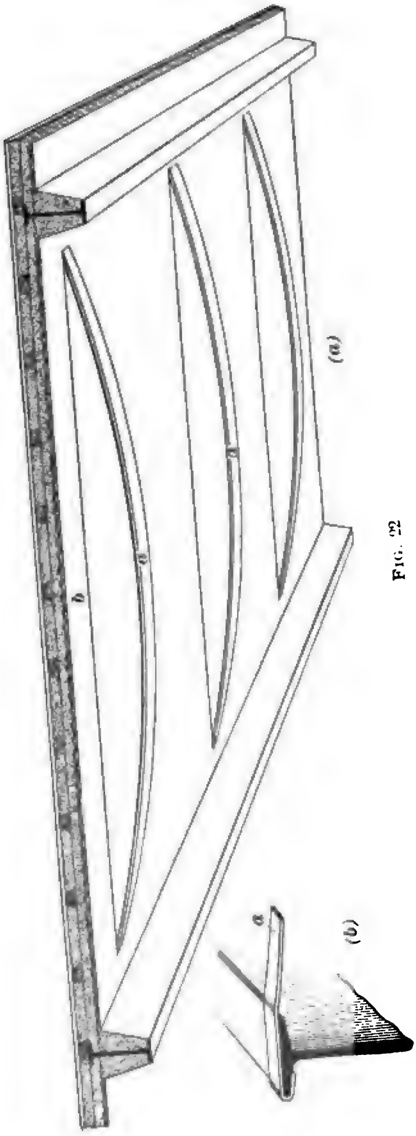


FIG. 22

For further finish and protection a flat ce
but the customary practice is to fireproof
arched channels separately, thus preservir
and allowing the maximum headroom. '
are usually spaced about 4 feet apart and
calculated. Tie-rods are required in th
panels and are conveniently placed over t
and laid between their flanges.

35. The **Suspension system,** shov
is unsightly when compared with the G
requires a flat or suspended ceiling wher
be considered. It consists of the us
reenforced by the iron or steel straps *a*, *a*,
or hooked to the upper flanges of the floo
at (*b*). In fact, it is just the reverse of th
Fig. 21, and as the slab *b* of concrete is
must be proportioned to resist that stre
when properly designed, develops great s
of 20 feet have been tested and found t
600 pounds per square foot.

36. Different investigations as to the
forced concrete slabs have led to man
has been generally agreed by the inve
metal embedded in the lower portion of t
the transverse strength to a considerable ex
of these investigations and tests have pr
that for the usual spans, that is, from 4 to
saving of from 300 to 400 per cent. in the dep
The great economy in the use of this con
appreciated when it is considered that a 4-

span for either safety or economy was supposed to be limited to 5 feet, but as the demands for greater spans became frequent, the possible span was lengthened from 5 feet to 8 feet by conservative engineers, and to 15 feet and 20 feet by the more venturesome. Even spans as great as 20 feet, when properly proportioned, are comparatively thin and are wonderfully economical when their strength is considered.

37. Cement concrete is not wholly unchanged by heat, but when the floor is well proportioned and the entire mass is securely bonded and tied with the expanded metal, as in this system, repeated tests have shown the construction to be thoroughly fireproof and capable of withstanding the heat of a severe conflagration.

There has been some question in the minds of engineers regarding the durability of this type of floor construction, owing to the possibility of the embedded steel corroding and thus diminishing the strength of the construction. Such doubts can hardly exist in view of the past experiments and experiences. Iron straps embedded in concrete for more than 20 years have been found to be in practically the same condition as when put in place, and there have been reports of medieval and ancient buildings in which iron embedded in mortar and cement for centuries has been found to be in a perfect state of preservation. The mortar or cement differed somewhat from the same materials used at the present day, but had practically the same nature.

38. Considering its strength, expanded-metal construction is as light as any floor system on the market; its weight for different thicknesses and spans is given in Table V. The typical sections assumed in compiling the table for both the flat and elliptical-arch construction are shown in Figs. 23 and 24, respectively, each illustration consisting of a transverse and a longitudinal section. The values given in this table do not include the weight of the flooring, as this will vary in the different classes of buildings.

39. Many tests have been made to ascertain the supporting strength of this floor construction, but in all cases the

TABLE V

APPROXIMATE WEIGHT FOR FLAT AND ELLIPTICAL MONOLITHIC FLOOR CONSTRUCTION

Type of Arch	Span of Arch Feet	Depth of I Beams Inches d	Thickness at Top of Arch Inches m	Weight per Square Foot of Cinder-Concrete Arch, Exclusive of Ceiling, Filling, and Sleepers Pounds	Weight per Square Foot of Stone-Concrete Arch, Exclusive of Ceiling, Filling, and Sleepers Pounds	Total Weight per Square Foot of Cinder-Concrete Arch Pounds	Total Weight per Square Foot of Stone-Concrete Arch Pounds
Flat Arch With Concrete Filling Around I Beams and With Suspended Ceiling	4	10	2	27.1	45.4	44.1	62.4
			3	32.	54.8	49.	71.8
	5	12	3	33.5	57.7	50.5	74.7
			4	38.3	67.3	55.3	84.3
	6	15	3	35.9	62.1	52.9	79.1
			4	40.8	71.6	57.8	88.6
			5	45.6	81.2	62.6	98.2
	7	15	4	39.8	69.6	56.8	86.6
			5	44.8	80.8	61.8	97.8
	8	18	4	42.	73.7	59.	90.7
			5	47.	83.5	64.	100.5
			6	52.	93.3	69.	110.3
Elliptical Arch Without Suspended Ceiling	4	10	2	36.1	63.1	46.1	73.1
			3	40.1	70.9	50.1	80.9
	5	12	2	39.6	69.5	49.6	79.5
			3	43.5	77.1	53.5	87.1
			4	47.7	85.3	57.7	95.3
	6	15	3	48.3	86.0	58.3	96.0
			4	52.6	94.5	62.6	104.5
	7	15	3	47.7	84.7	57.7	94.7
			4	52.1	93.2	62.1	103.2
			5	56.2	101.2	66.2	111.2
	8	18	4	57.	102.5	67.	112.5
			5	61.2	111.0	71.2	121.
	9	18	4	56.1	101.2	66.1	111.2
			5	60.4	109.7	70.4	119.7
		20	6	68.2	124.3	78.2	134.3

results have been satisfactory. The data relative to several tests are given in Table VII. The conditions of these tests are more severe than would occur in actual practice, because in several cases the slab to be tested was an isolated section of flooring supported on only two edges. In other cases, while the slab of concrete or the arch was supported on four edges, it was not, except in the test of the elliptical arch, a

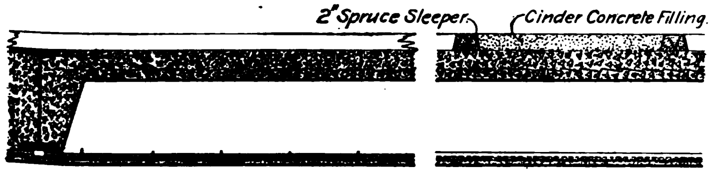


FIG. 23

part of a continuous floor system and, hence, the advantage of a continuous beam was lost.

40. Table VI will be found convenient in obtaining the allowable load on the expanded-metal system of flooring for both uniformly distributed and concentrated loads. This table, though compiled by the manufacturers of expanded metal, is conservative, and the loads given, when used with

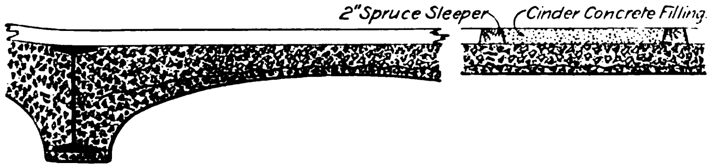


FIG. 24

a factor of safety from 4 to 6, may be considered perfectly safe for the floor system. The values in the table are based on the assumption that the concrete is at least 1 month old and is made in the proportion of one part of cement, three parts of sand, and six parts of cinder or stone. Portland cement is used in this construction and the concrete is machine mixed.

TABLE VI

BREAKING LOADS OF CONCRETE FLOORS

Type of Floor	Thickness of Slab	Span in Feet: 4		5		6		7		8		9		10		Moment of Resistance for Concentrated Loads
		Uniform Load per Square Foot	Concentrated Load	Uniform Load per Square Foot	Concentrated Load	Uniform Load per Square Foot	Concentrated Load	Uniform Load per Square Foot	Concentrated Load	Uniform Load per Square Foot	Concentrated Load	Uniform Load per Square Foot	Concentrated Load	Uniform Load per Square Foot	Concentrated Load	
	Inches	Pounds	Tons	Pounds	Tons	Pounds	Tons	Pounds	Tons	Pounds	Tons	Pounds	Tons	Pounds	Tons	
Cinder Concrete With Expanded Metal of No. 10 Gauge, 2½-Inch Mesh	2	765	.51	460	.41	300	.34									12,260
	2½	1,190	.79	700	.63	465	.52	330	.45							18,940
	3	1,680	1.12	1,000	.90	660	.74	468	.64	350	.56					26,800
	3½	2,250	1.50	1,350	1.20	885	1.00	628	.86	468	.75	360	.67			36,000
	4	2,860	1.90	1,710	1.52	1,130	1.27	795	1.09	592	.95	458	.85	365	.76	45,700
	4½	3,320	2.22	1,980	1.77	1,300	1.47	920	1.26	690	1.10	530	.98	425	.88	53,000
	5	3,720	2.48	2,220	1.98	1,460	1.66	1,040	1.42	775	1.24	600	1.10	480	.99	59,600
	5½	4,150	2.77	2,480	2.22	1,640	1.84	1,150	1.58	860	1.38	665	1.23	530	1.11	66,400
	6	4,550	3.02	2,720	2.43	1,790	2.02	1,270	1.73	950	1.52	730	1.35	582	1.22	72,900
Cinder Concrete With Expanded Metal of No. 10 Gauge, 3-Inch Mesh	2	820	.55	492	.44	324	.36									13,140
	2½	1,280	.86	768	.68	505	.57	357	.49							20,500
	3	1,820	1.22	1,090	.97	715	.81	509	.69	380	.61					29,100
	3½	2,440	1.63	1,460	1.30	960	1.09	680	.93	504	.81	392	.72			39,000
	4	3,130	2.08	1,870	1.67	1,230	1.39	872	1.19	650	1.04	502	.93	333	.83	50,000
	4½	3,880	2.58	2,320	2.07	1,530	1.72	1,080	1.48	807	1.29	622	1.15	413	1.03	62,000
	5	4,660	3.12	2,800	2.49	1,840	2.07	1,300	1.78	975	1.56	750	1.38	500	1.25	74,800
	5½	5,440	3.63	3,260	2.90	2,140	2.42	1,520	2.07	1,130	1.81	875	1.61	580	1.45	87,000
	6	6,000	4.00	3,600	3.19	2,340	2.66	1,680	2.27	1,250	2.00	960	1.77	640	1.60	95,800

Rock Concrete With Expanded Metal of No. 16 Gauge, 2½-Inch Mesh	2	1,065	.71	638	.57	420	.48									17,100
	2½	1,600	1.07	958	.85	630	.71	445	.61	333	.51					25,620
	3	2,210	1.48	1,320	1.18	870	.98	620	.84	460	.74	355	.65			35,400
	3½	2,720	1.82	1,630	1.45	1,070	1.21	760	1.04	565	.91	436	.81	347	.72	43,500
	4	3,150	2.12	1,890	1.68	1,245	1.40	878	1.20	655	1.05	506	.93	403	.84	50,500
	4½	3,600	2.40	2,150	1.91	1,410	1.60	1,000	1.37	750	1.20	576	1.06	460	.96	57,500
	5	4,040	2.69	2,420	2.15	1,590	1.79	1,120	1.54	840	1.35	646	1.20	516	1.08	64,500
	5½	4,470	2.98	2,670	2.38	1,762	1.99	1,240	1.70	930	1.50	716	1.32	571	1.19	71,500
	6	4,900	3.28	2,930	2.62	1,930	2.18	1,375	1.87	1,020	1.64	786	1.45	628	1.31	78,500
Rock Concrete With Expanded Metal of No. 10 Gauge, 3-Inch Mesh	2	1,165	.77	695	.62	438	.52	324	.44							18,640
	2½	1,790	1.19	1,070	.96	710	.80	500	.68	375	.60					28,700
	3	2,500	1.66	1,500	1.34	984	1.11	695	.95	520	.83	402	.74	320	.67	40,000
	3½	3,250	2.16	1,940	1.74	1,280	1.44	900	1.24	675	1.08	522	.96	415	.87	52,000
	4	4,100	2.72	2,450	2.18	1,610	1.82	1,140	1.56	850	1.36	656	1.21	522	1.09	65,400
	4½	4,750	3.10	2,840	2.53	1,870	2.11	1,320	1.81	990	1.58	763	1.41	610	1.26	76,000
	5	5,320	3.54	3,180	2.84	2,090	2.36	1,480	2.03	1,110	1.78	854	1.58	680	1.42	85,000
	5½	5,900	3.93	3,520	3.15	2,320	2.62	1,640	2.25	1,230	1.97	950	1.75	755	1.57	94,400
	6	6,450	4.30	3,860	3.45	2,530	2.87	1,800	2.46	1,345	2.15	1,040	1.91	825	1.72	103,200

TABLE VII

LOAD TESTS OF CONCRETE AND EXPANDED-METAL ARCHES

Character of Arch	Number of Supporting Sides	Character of Load	Total Area Covered by Load Square Feet	Total Amount of Load Pounds	Total Load per Square Foot of Floor Area Pounds	Deflection Inches
Flat arch shown in Fig. 23, described in Art. 33; span 6 feet, thickness 3½ inches, gauge No. 12, mesh 2½ inches.	3	Bricks laid close and bonded	75	83,000	1,1[illegible]	[illegible]
Flat arch shown in Fig. 23, described in Art. 33; span 7 feet [illegible] inches, thickness [illegible] inches, rock concrete.	2	Pig iron piled close			750	[illegible]
[illegible] arch shown in Fig. [illegible], described in Art. 33; span [illegible] feet, [illegible]	4	Pig iron piled [illegible]	10	11,000	1,100	[illegible]

ROEBLING SYSTEM: ARCH CONSTRUCTION

41. The **Roebling system** of fireproof-floor construction embodies essentially the same materials as the system using expanded metal. It consists of concrete supported by woven-wire lathing embedded in its lower surface. In this system, cinder concrete is used entirely and is composed of either one part of high-grade Portland cement, two parts of clean, sharp sand, and five parts of steam ashes, or one, two and one-half, and six parts, respectively, of the same materials.

It differs from the monolithic construction, which uses expanded metal, in that the woven wire is not designed to act as a reenforcing member to the concrete but is used as a permanent center upon which the concrete can be laid during construction, in the form of either a flat or segmental arch. Many types of floor construction have been designed by the manufacturers to meet the numerous conditions encountered in building practice.

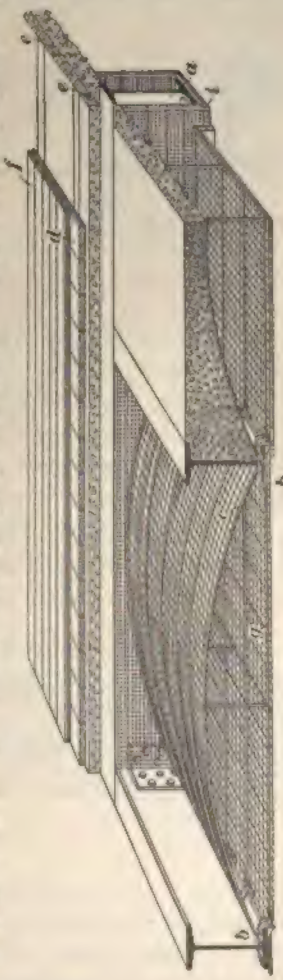


FIG. 25

42. Fig. 25 shows the typical construction employed in the Roebling system. In this figure, *a* is the principal girder in the floor construction and supports the auxiliary girders *b*, *b*, between which is sprung woven cloth reenforced by light $\frac{3}{16}$-inch steel rods *c*, *c* woven in every 9 inches. The reenforced

wire cloth forms a permanent center on which the cinder concrete may be placed. On the concrete floor so constructed, the sleepers e, e, which are about 2 inches by 3 inches, are laid and filled in between with some poor concrete or other fireproof filling. On the sleepers the rough flooring d is laid which provides a level surface for the finished flooring f.

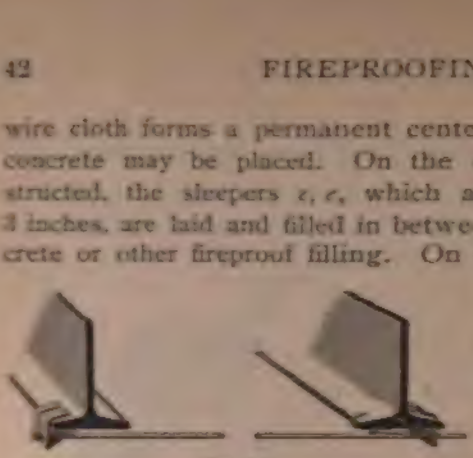


FIG. 26

The flat ceiling beneath this arch construction is composed of plaster placed upon woven-wire lathing. The woven wire for the ceiling is supported upon $\frac{5}{16}$-inch round rods g pulled tight between the lower flanges of the floorbeams. These rods are placed 18 inches from center to center, and are secured to the lower flange of the beams by the forged steel clip shown in Fig. 26. The ceiling is securely supported at the center by suspension rods from the crown of the permanent centering. It is common in such construction to clamp light bar-iron furring around the lower flange of the main girder, as shown at i. The woven-wire cloth used for the ceiling is placed upon this furring, and the space between it and the beam is filled in with cinder concrete. The woven

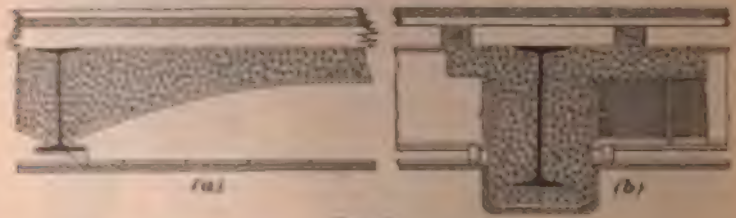


FIG. 27

wire, both in the permanent centering and in the ceiling construction, is securely laced to the rods forming the main support. The details of construction are shown in Fig. 27, in which (a) is a transverse section of the arch and (b) one of the principal girders. This construction is known as **System A, Type 1**.

43. It is sometimes desirable, where deep girders are used and a pleasing finish is desired, to make the girder protection in the form of a cornice, as in Fig. 28. A framework is constructed of light iron strips *a* bent to the desired outline, as shown in (*a*). This framework is attached to the

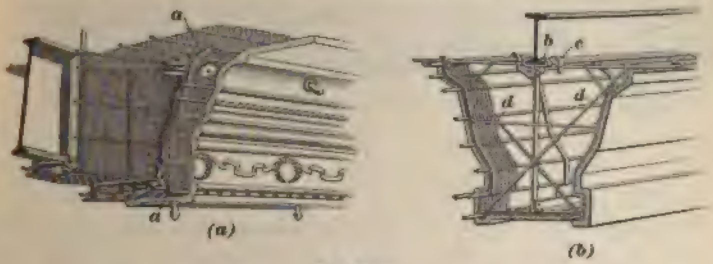


FIG. 28

girder by means of the clamps *b*, *c*, as shown at (*b*), and is then covered with wire lath, upon which the plastering is applied. Iron bars *d* are introduced in the frame at the angles to brace the bent iron strips.

44. For factory, store, and warehouse construction, where great strength is required and economy of space is important, and in such buildings as are repositories for large quantities of inflammable goods, a construction must be used that thoroughly protects the steel beams and furnishes great resistance to heavy statical loads. The Roebling construction for such buildings is shown in Fig. 29 and is designated

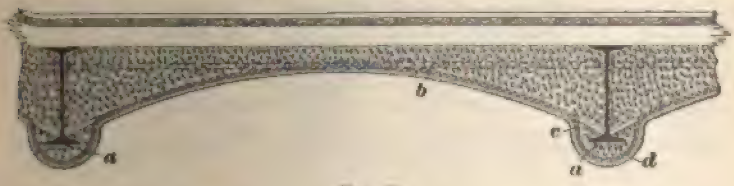


FIG. 29

as **System A, Type 2.** In this figure, *a*, *a* are the secondary beams of the floor construction, tied together by the tie-rods at *b*. The usual reenforced woven-wire cloth is sprung from the lower flanges of the steel beams. These reenforced rods, in heavy construction, are made $\frac{9}{16}$ inch in diameter.

To secure a concrete protection for the lower flanges of the beams, the woven wire is omitted for a distance of from 4 to 6 inches on each side of the web of the beam and the ribs allowed to project as shown at *c*. A separate form of wire lathing is bent around the bottom flanges of the steel beams

FIG. 30

and laced to the ribs of the centering, as shown at *d*, and when plastered over gives the appearance of a ribbed or groined ceiling. The principal reasons for bending the woven-wire lathing around the lower flange of the beam in this way are to afford protection from fire and to provide a neat finish at the spring of the arches.

The sleepers may be placed upon the top of the cinder concrete filling, or they may be dropped below the top flange of the beam, as designated in Fig. 30 by a transverse and longitudinal section, which is also called System A, Type 2. When this method of construction is employed, the head-room required by the floor system is lessened the depth of the sleepers, which is a valuable consideration where there are many floors, as 2 or 3 inches saving in one floor means a reduction of several feet in the height of the building.

FIG. 31

45. For department stores, halls of record, banks, and libraries, the type of construction shown in Fig. 29 is often employed. It is, however, usually necessary in such buildings to provide a flat ceiling, in which case the construction shown in Fig. 31, known as **System A, Type 3,** is adopted.

In this construction, the ceiling is supported on $1\frac{1}{2}'' \times \frac{3}{16}''$ merchant bar iron, suspended from the lower flange of the floorbeams by wrought-iron clamps *a*, as shown in the transverse section at (*a*), and also in the longitudinal section at (*b*). These pieces *b* of bar iron are placed on edge about 16 inches from center to center. To these bars, No. 20 woven-wire lathing is laced with No. 18 galvanized-wire lacing, shown at *c*. Where a protecting ceiling of this nature is suspended below the floor arch, the tie-rods *d* may be placed close to the spring of the arch, projecting through the concrete, which leaves them exposed, being protected only by the ceiling. It is doubtful whether the ceiling affords adequate protection to the tie-rods, so that when they form an important member of the construction they should be protected by concrete filled in around them, which is supported by channels provided in the woven-wire centering.

When the floorbeams are more than 10 feet apart, instead

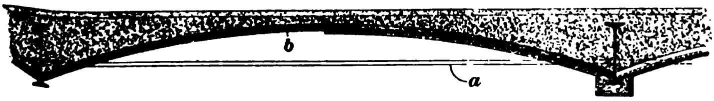


FIG. 32

of using the woven-wire cloth with reenforcing rods alone, it is necessary to introduce $2'' \times 3'' \times \frac{3}{16}''$ **T** irons *b*, as shown in Fig. 32, placed at 24-inch centers and held laterally by $\frac{1}{2}'' \times \frac{1}{4}''$ steel spacers or separators. This construction is known as **System A, Type 4.** The **T** irons are of suitable sectional area to support any load that the floor is likely to be required to sustain, and the woven-wire lath containing the stiffening ribs is laced to their under side with galvanized wire. This construction, as in the previous case, acts as a permanent centering besides reenforcing the arch to a considerable extent. The tie-rods *a* in this system are exposed, and should be placed the usual distance apart, that is, about eight times the depth of the beam, for spans not over 12 feet. Where the spans are greater than this, the tie-rods should be especially designed to sustain the thrust of the arch.

This style of floor has been successfully employed for

spans as great as 18 feet. The objectionable feature consists in the fact that the tie-rod is exposed, but this disadvantage may be overcome by securing light bar-iron furring upon the rod at intervals and turning woven-wire lath around this skeleton construction. The form thus made is filled with concrete and the outside is plastered. The appearance of the ceiling is not materially affected by this means and a thoroughly fireproof system is attained.

ROEBLING SYSTEM: FLAT CONSTRUCTION

46. The flat system of construction, involving the use of bar iron to reenforce the concrete and of woven-wire lathing to supply a permanent centering, has been designed to meet the requirements for a light, low-priced floor adapted to a wide spacing of the beams. The flat system of construction is known as the **Roebling flat construction System B,** and is shown in detail in Fig. 33. It consists of a light iron framework embedded in concrete and spanning the interval

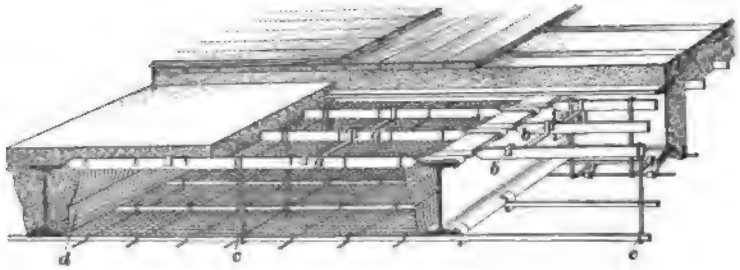


FIG. 33

between the steel beams, thus forming a monolithic slab reenforced by wrought-iron tension members. The light iron framework consists of flat iron or steel bars *a* set on edge and spaced 16 inches between centers. The flat bars are given a quarter turn at both ends so that they will lie flat on the top flange of the steel beams supporting the floor system, and the turned ends form a hook, or clamp, that securely attaches them to the upper flange, thus obviating the necessity of tie-rods. These pieces of bar iron are held in

position, laterally, by flat or half-round iron spacers, having a hook *b* turned on the end. The spacers are placed at suitable intervals to separate and brace the bars. The woven-wire lathing, with the $\frac{1}{4}$-inch round stiffening ribs woven in every $7\frac{1}{2}$ inches, is applied to the under side of the bars, the stiffening ribs being run crosswise over the bars and laced to them at every intersection. On the wire lathing supported in this manner, cinder concrete is deposited, thoroughly embedding the light ironwork.

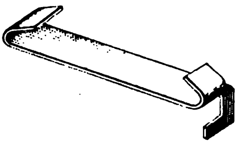


Fig. 34

The ceiling used in conjunction with this system of construction is the same as that employed with the arch construction, but as the span is greater in this system, $\frac{3}{4}'' \times \frac{3}{16}''$ flat bar iron set on edge is used for the support of the ceiling, instead of $\frac{7}{16}$- or $\frac{9}{16}$-inch round. The flat bars are clamped to the lower flange of the I beam by means of a wrought-iron clip, shown in detail in Fig. 34. When the span is great, that is, over 8 feet, the ceiling is supported by wire *c*, *c* from

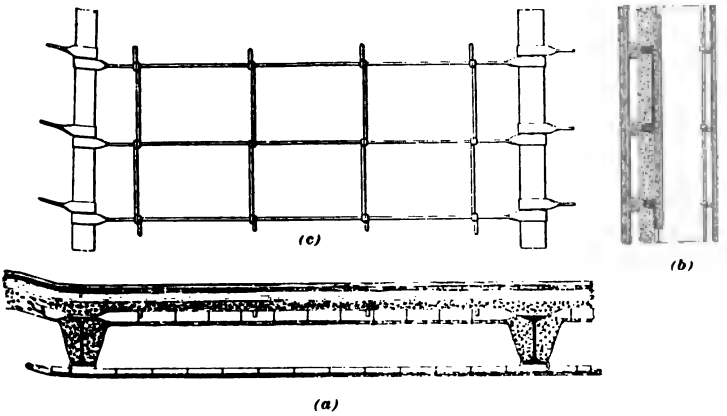


Fig. 35

the rods forming the tension members in the floor system. The I beams in this construction are protected by means of concrete filling, held in place by the upper and lower flanges, and by woven-wire lathing, put in place as shown at *d*.

47. The type of floors used for public buildings, offices, theaters, churches, schools, hospitals, hotels, etc. is known as **System B, Type 1**, and is designated in Fig. 35, in which (*a*) and (*b*) show longitudinal and transverse sections of the floor, while (*c*) is a plan showing the arrangement of the flat bars and the steel-rod spacers.

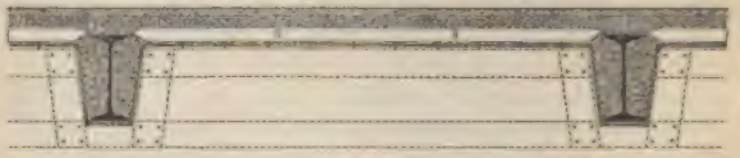


FIG. 36

System B, Type 2, is shown in Figs. 36 and 37. In Fig. 36, the dotted lines indicate the temporary wood centering that may be used in place of the permanent wire centering.

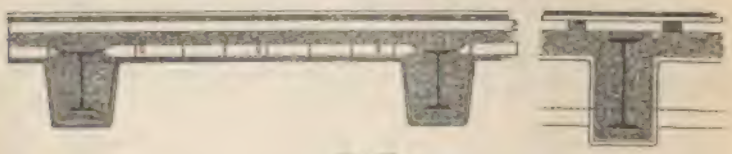


FIG. 37

The advantage of the wood centering lies in the fact that when a number of men are at work at other trades on the same floor, the concrete will not be greatly disturbed should

FIG. 38

they step on it. In Fig. 37 is shown, in transverse and longitudinal sections, a construction that is adaptable for warehouses, depots, stores, machine shops, and factories.

The construction shown in Fig. 38 is designated as **System B, Type 3**, and is used for department stores, libraries, etc.

Where the headroom is small and where it is desired to keep the distance from the top of the finished floor to the under side of the ceiling as small as possible, the flat system may be used to advantage. This is illustrated in **System B, Type 4**, shown in Fig. 39, where it may be seen how, by

FIG. 39

ingenious arrangement and design, this type of floor construction can be kept to a minimum thickness, the over-all dimension from the finished floor to the under side of the ceiling being 9 inches. The type of floor shown in this figure is especially adaptable for hotels, apartment houses, hospitals, and residences.

48. Where the span between the steel beams is over 9 or 10 feet, the construction shown in Fig. 40, known as **System B, Type 5**, will be found very economical. It is similar in every way to the construction just described, with the exception that the flat bar iron is bent downwards at the center and is embedded in the concrete in this position. The strength of this floor construction depends entirely on the tensile strength of the rods, for the span is so great that a monolithic slab from girder to girder is of little value. It is, however, sufficient to carry the load between the steel rods, which are spaced about 12 inches from center to center. This type of construction is particularly adapted for floors

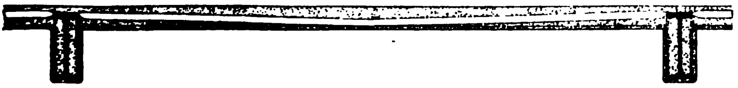


FIG. 40

supporting light weights and where economy is an important consideration. It has been used for spans as great as 22 feet, but the most economical results are obtained when the girders are spaced not farther apart than 16 feet. The weight of the concrete and embedded steel bars, constructed

as shown in Fig. 40, is about 43 pounds per square foot; an additional 7 pounds must be added if two coats of plaster are used for the ceiling and around the steel beams.

49. The Roebling system of fireproof-floor construction ranges in weight from 28 to 50 pounds per square foot of floor surface, as will be observed from Table VIII. In this table, the weights are given for the concrete and wire alone, and the concrete is considered as being level with the top flange of the beams; the weight of the ceiling is not included.

TABLE VIII

WEIGHT OF ROEBLING SYSTEM OF CONCRETE-AND-IRON FLOORS

System A					System B				
Height of Concrete Leveled Above Under Side of Floorbeams	Maximum Spacing of Floorbeams		Thickness of Crown at Center of Arch	Weight, per Square Foot, Including Only Concrete and Wire	Type of Construction	Spacing of Beams	Depth of Beams	Thickness of Concrete	Weight, per Square Foot, of Concrete, Embedded Iron, and Wire
Inches	Feet	Inches	Inches	Pounds	Figure	Feet	Inches	Inches	Pounds
8	4		3	33	36	8	10	3½	30
9	4	6	3	34	37	5	10	3½	35
10	5		3	36	38	5	15	3½	50
12	6		3	41	39	6	6	3½	28
15	7	6	3	47					

The ceiling in all cases weighs from 7 to 10 pounds per square foot, depending on whether two or three coats are used in plastering. In order to determine the entire dead load of this system of floor construction, to the loads given in the table should be added the weight pro rata per square foot of floor surface for the steel beams, wood, or other finished floor.

50. Numerous severe tests have demonstrated that the strength of the Roebling system of floor construction with

usual spans, is sufficient to safely carry a load of 300 pounds per square foot of floor surface, and it may be designed for the heaviest loads encountered in building construction.

The drop test, which is the most severe of all tests, was made at Boston, Mass., upon a floor constructed on 15-inch steel beams spaced 4 feet 6 inches apart; two tests were made, which are recorded by the company as follows:

1. A shot weighing 46 pounds was dropped from a height of 43 feet 6 inches, striking fairly on a 3″ × 3″ sleeper embedded in the cinder concrete at the crown of the arch. The sleeper was crushed to splinters and the cinder concrete between the sleepers was broken. The sleeper and concrete were then removed and the shot was again dropped from the same height. It struck directly on the crown of the unprotected arch at its thinnest point, which was approximately 2½ inches in thickness. The effect of the impact was to embed the shot one-third its diameter in the concrete. A proportionately small, round protuberance appeared on the under side of the arch; otherwise, the arch was apparently uninjured. There were no cracks. The blow was equal to 2,000 pounds at 1 foot fall.

2. A mass of concrete weighing 410 pounds, which was being removed to make an aperture for a light well, was allowed to fall from the story above, a distance of 11 feet, to the concrete arch below. The shock did not even crack the arch.

Besides these tests, numerous others have been made from time to time, both with concentrated and with uniformly distributed loads, with the results given in Table IX. In making the tests, especially where bricks were used in loading, great care was exercised to prevent the bonding of the load, which would give incorrect results, since much of the test load would be transmitted to the beams instead of being supported by the arch.

FIREPROOFING

(PART 2)

FLOOR CONSTRUCTION—(Continued)

THE COLUMBIAN SYSTEM

1. A system of monolithic floor construction known as the **Columbian system** consists of concrete slabs reenforced and supported in position by special rolled steel bars, which usually have the sectional form shown in Fig. 1. These bars are either framed to the steel beams, as in Fig. 2 (*a*), or are supported by special clips or stirrups hung over the top flange of the steel beam, as in Fig. 2 (*b*). In view (*a*), the special steel bars are framed to the steel girders, as at *b*. Under the steel bars, a wooden center is placed on which the concrete filling is tamped. When the concrete has set, it forms practically a stone slab supported by the light steel beams that are thoroughly protected by the concrete; any character of finished floor can be used on it. In view (*b*), steel bars are hung from the top flange with a stirrup *c*. The use of this stirrup, which is shown in detail in Fig. 3, eliminates the work required for riveted or bolted connections and permits the bars to be more readily placed in position. The paneled-ceiling treatment is shown in Fig. 4, but where a flat ceiling is required, the construction shown in Fig. 5 is employed.

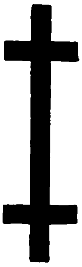


FIG. 1

2. In the construction shown in Fig. 5, which is known as *double construction*, the floor system is practically the same

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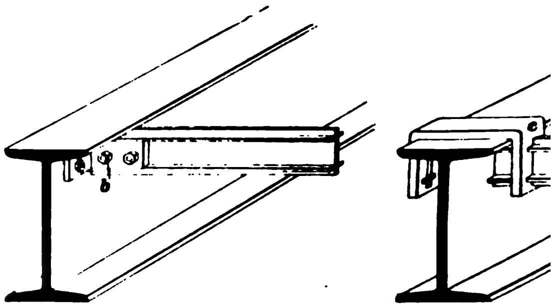
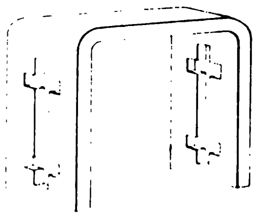
omitted, as the suspended ceiling furnishes
tection. The ceiling is constructed in the
the floor; that is, special rolled bars are s



(a) (b)

FIG. 2

lower flanges of the I beams and are emb
layer of concrete, which they support. The
is formed by plastering the bottom surface
slab. The principal beams or girders are
construction shown in Fig. 4 by an especia
crete slab *b*, so arranged as to provide an
lower flange of the girder, thus providing
protection and having iron anchors *c* embed



it in place. This speci
detached below the lef
Fig. 4, and in place at
beam. The embedded
are turned over the low
girder and hold the sla

3. All these types

the lower flange of the steel girder by the wrought-iron clamp and hanger *b*. After the steel bars forming the main support of the ceiling are in position, the concrete *c* is dumped upon this centering and tamped in place. After the concrete

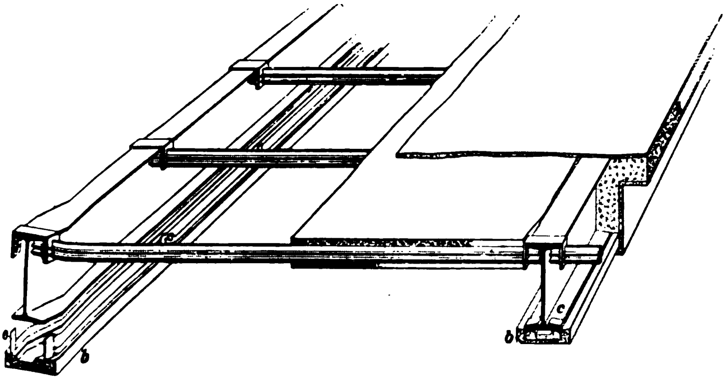


FIG. 4

forming the ceiling has obtained its initial set, the sleepers *d*, which carry the wooden supports for the centering upon which the floor is constructed, are put in place. The centering for the construction of the floor is removed through openings

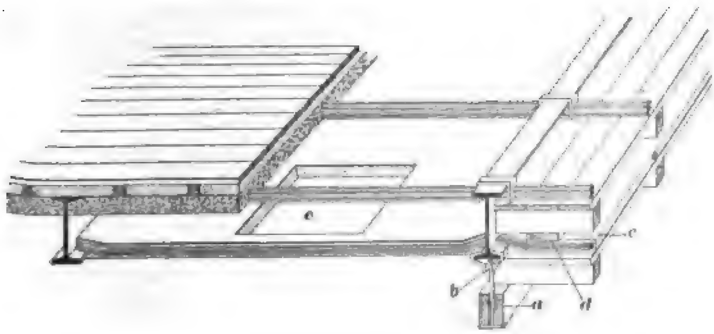


FIG. 5

provided at intervals in the concrete ceiling, the openings being afterwards closed with a concrete slab. In Fig. 5, *c* shows one of these openings, which is rectangular and is provided with beveled sides so that the concrete slab can be

introduced through it and, on being lowered into place, make a tight and secure ceiling.

4. Instead of the special steel bars, shallow I beams are occasionally used, but owing to the fact that the special rolled sections can be obtained in numerous sizes and are especially adapted for use in connection with concrete con-struction, a considerable saving is often effected by their use where the distance between the girders is not over 7 or 8 feet. The most economical arrangement for the floor system is to place the steel girders from 7 to 8 feet apart for the panel construction shown in Fig. 4, and from 6 to 7 feet for the double construction shown in Fig. 5. If the spans are increased beyond these limits there is little saving b using this system of construction, as the ordinary I beam rolled by the mill can be economically substituted for th special sections.

The span recommended for the Columbian system is fro 6 to 7 feet and the steel girders should be arranged for tl construction at these distances. In this construction no ti rods are used and consequently the cost of punching a other shop work on the steel girders and beams is consid ably reduced. The concrete used is composed of Portla cement, sharp sand, and crushed furnace slag or broke stone screenings, though for light floors, cinder concrete commonly used.

TABLE I

WEIGHT PER SQUARE FOOT OF COLUMBIAN SYSTEMS

Style of Ceiling	Size of Steel Bars Inches	Weight of Ceiling Stone Concrete Pounds	Weight of Ceiling Cinder Concrete Pounds
Paneled . . .	1	43.5	22.7
Paneled . . .	2	47.4	24.8
Paneled . . .	2½	55.6	30.4
Flat	1	50.0	31.7
Flat	2	55.1	34.7
Flat	2½	61.3	37.3

NOTE.—Exclusive of the beams, for the weight of the fill between the sleepers, or wooden floor, add 6 pounds per square foot per inch thickness of fill.

TABLE II

RESULTS OF TESTS MADE ON THE COLUMBIAN FLOOR SYSTEM

Span Feet	Span Inches	Character of Load	Size of Bar Inches	Spacing Feet	Spacing Inches	Load per Square Foot Pounds	Deflection of Beam Inches
16	0	Bricks, and bags filled with sand, uniformly distributed	4¼	2	00	300	0
						350	1/8
						450	1/8
						500	3/16
						600	1/4
						700	3/8
						900	7/16
16	0	Central load of bricks				1,000	No signs of failure
6	6	Pig iron distributed .			20	1,400	
5	0	Uniformly distributed	2	2	00	1,000	
8	0	Uniformly distributed	2½		20	810	
8 to 9	0	Uniformly distributed	2		22	1,000	
7	0	Uniformly distributed	2	2	00	820	
16	0	Uniformly distributed	3½	2	00	350	
6 to 7		Uniformly distributed				600	

tension member and supplies the tensional resistance that the concrete lacks, as explained in connection with expanded metal construction, in *Fireproofing*, Part 1.

The most instructive of the numerous tests that have been made of this system are given in Table II. The tests in all cases have been made as nearly as possible under conditions that would actually exist in buildings.

7. The safe live loads uniformly distributed, allowing a factor of safety of 4, that the Columbian system will sustain, are given in Tables III and IV. The ribbed steel bars are considered as spaced at the maximum distance of 2 feet and are of the sections shown in Fig. 6 (*a*), (*b*), (*c*), and (*d*), which represent sections of the $3\frac{1}{2}$-, $4\frac{1}{4}$-, 5-, and 6-inch bars, respectively. Table III gives the entire safe live load upon a single floor panel, in tons, while Table IV gives the safe live load per square foot of floor surface, in pounds.

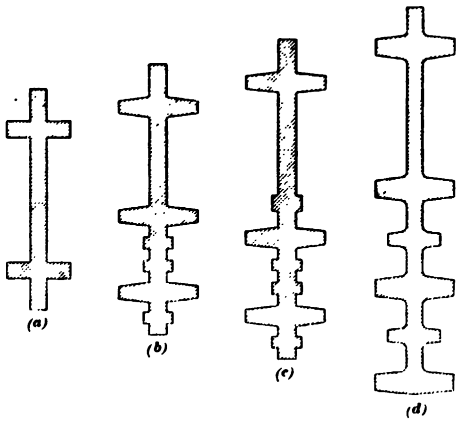


FIG. 6

8. The essential advantages of the Columbian system of fireproof-floor construction are that the steelwork is completely encased in a monolithic slab of refractory material and that the steel rolled shapes, which are the auxiliary supports of the floor system, are especially designed for great strength and are economical when placed at limited distances apart. The principal girders are completely protected with concrete filling between the flanges and by the especially molded slab of concrete, which provides an air space and is attached to the lower flange. The floor system is shallow and takes up little headroom, and at the same time it is strong enough for all building

TABLE III

SAFE LOAD UNIFORMLY DISTRIBUTED ON COLUMBIAN FLOOR SYSTEM

Distance Between Supports Feet	Thickness of Concrete. Inches				Distance Between Supports Feet	Thickness of Concrete. Inches		
	7½	6½	5¾	5		4	3½	3
	Size of Ribbed Bars Spaced 2 Feet Between Centers. Inches					Size of Ribbed Bars Spaced 2 Feet Between Centers. Inches		
	6	5	4¼	3½		2½	2	1
	Load per Panel. Tons					Load per Panel. Tons		
12	4.95	4.35	3.75	3.30	5	2.00	1.70	1.45
13	4.50	4.00	3.45	3.05	6	1.65	1.45	1.20
14	4.15	3.65	3.20	2.80	7	1.40	1.25	1.00
15	3.85	3.40	3.00	2.60	8	1.20	1.10	.90
16	3.60	3.20	2.80		9	1.00	.90	
17	3.40	3.00			10	.80	.70	
18	3.20	2.70			11	.60		
19	2.65							
20	2.00							

TABLE IV

SAFE LOAD UNIFORMLY DISTRIBUTED ON COLUMBIAN FLOOR SYSTEM

Distance Between Supports Feet	Thickness of Concrete. Inches			
	7½	6½	5¾	5
	Size of Ribbed Bars Spaced 2 Feet Between Centers. Inches			
	6	5	4¼	3½
	Load per Square Foot. Pounds			
12	412	362	312	275
13	346	306	265	235
14	296	260	230	200
15	256	226	200	175
16	225	200	175	
17	200	175		
18	177	150		
19	140			
20	100			

Distance Between Supports Feet	Thickness of Concrete. Inches		
	4	3½	3
	Size of Ribbed Bars Spaced 2 Feet Between Centers. Inches		
	2½	2	1
	Load per Square Foot. Pounds		
5	400	340	290
6	275	242	200
7	200	178	140
8	150	135	112
9	112	100	
10	80	70	
11	55		

purposes. That the system offers adequate fire-protection has been proved by a number of tests.

9. Fire-tests of the Columbian floor were made by the Department of Buildings in the city of New York in 1897. The usual enclosure, somewhat similar to that described in Art. 15, *Fireproofing*, Part 1, was constructed. Near the bottom of the structure two sets of grate bars, one 18 inches above the other, were provided, covering the entire floor area, so that the air entering at the bottom would be heated before passing through the principal bed of fuel, which was of hard wood. The draft was obtained by a chimney 15 inches square extending 6 feet above the roof. The floor over the enclosure was constructed in three panels supported on 10-inch I beams, placed 4 feet from center to center. The special ribbed bars used were 2 inches deep. They were placed 20 inches between centers and were suspended from the upper flange of the steel beams, or girders. A 3-inch bed of concrete, composed of one part of Portland cement, two parts of sand, and five parts of broken stone, was so laid as to thoroughly encase the bars and to come level with the top of the 10-inch beams. The lower flange of the I beam was protected by the monolithic slab construction, as described in Art. 2. On the top of the concrete forming the floor were placed 2″ × 3″ sleepers 16 inches from center to center; the spaces between these sleepers were filled with a poor concrete, composed of one part of cement, five parts of sand, and ten parts of stone screenings. The finished floor was nailed to these sleepers.

The tests were made 30 days after the construction of the floor; the center panel was loaded with 150 pounds per square foot of pig iron distributed as equally as possible. A fire being lighted, within 30 minutes a temperature of 1,800° was attained and a few minutes later 2,000° was reached; for 5 hours during the test the temperature was between 1,500° and 2,000°. At the end of this period water at a pressure of 60 pounds was applied to the under side of the floor for 15 minutes; the upper side was then flooded for 5 minutes.

On careful examination, after the test, the floor was found to be intact, though some of the slab casings on the lower flanges of the beams and a portion of the concrete underneath the bars in the center span had been washed off by the force of water. The deflection of the floor, measured before the application of the stream of water, was about $3\frac{1}{2}$ inches, but on cooling it was found to be 3 inches. The **I** beams supporting the floor had sustained a permanent deflection in the center equal to this amount.

FAWCETT FIREPROOFING SYSTEM

10. The **Fawcett system** of fireproofing bears a similarity to all systems using lintels as a means of carrying the floor construction between the steel girders, or beams. It is, however, distinguishable from all other systems of fireproof-floor construction in that it assumes to provide a circulation of air around the steel beams, which is intended to protect them from the heat of a conflagration. This system, as shown in Fig. 7, consists of hollow terra-cotta lintels supported upon the lower flange of the steel floor-beams. These lintels *a*, which have the cross-section shown in (*a*), are placed in such a manner that their shortest diagonal *b b* in (*b*) is at right angles to the web of the steel beams. Since the tiles are laid in this manner, a special tile is needed at the end of each bay. This tile, shown in section in (*a*), consists of the ordinary form of tile cut on the diagonal in the molding, and is called a *split tile*. It is provided with a lip *c* that rests upon the adjacent lintel.

The tiles are composed of dense terra cotta, or pipe clay, molded to form, and burned to a considerable degree of hardness, so as to obtain the required transverse strength. After they are laid in position between the steel beams, concrete is tamped upon the top, as shown at *d*. The concrete takes a bearing on the lower flanges of the steel floor-beams and consequently carries the entire load of the floor, while the tile merely acts as a permanent centering, which, being hollow and provided with air spaces, forms an admirable

fire-resisting medium, besides lightening the floor construction to a great extent. The principal purposes of the tubular tile is to provide a circulation of cold air around the steel beams, which is done by introducing in the wall a hollow block *b*, Fig. 8 (*a*) and (*c*). This block is simply an iron

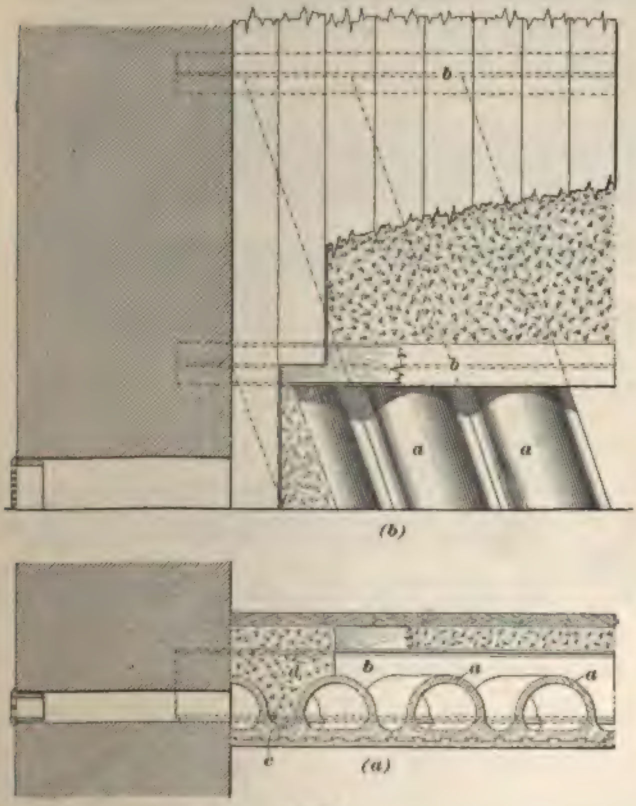


Fig. 7

thimble, or box, provided with a grating to prevent clogging with dust or rubbish and the entrance of birds. The cold air from the outside circulates through this opening and through the tubular tiles, as shown in (*a*) and (*b*), passing through the various tiles and along the entire length of the lower flange of the steel beam through the openings *g* in the ends

of the tile lintels. The lower part of the tile is formed wi dovetailed grooves, as shown in Fig. 7 (*a*), to secure a k for the finished ceiling.

Where the ends of the tiles abut a wall, a steel channel must be used as a support, as shown in Fig. 8 (*a*). Ca must be taken, in such a case, that the channels do n entirely close up the ventilation flues, the opening of whic

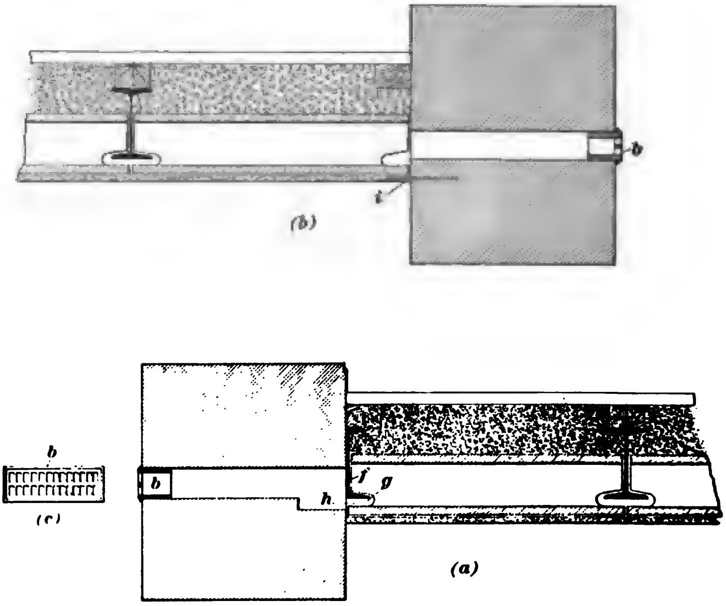


Fig. 8

must be dropped somewhat as shown at *h*. The use of a channel may be obviated by building into the brickwork a ½" × 6" plate *i*, as shown in (*b*), which should project sufficiently for the end tiles to take a bearing. At the end of the bay, where the walls run approximately parallel with the length of the tile, no channel or plate is required, for the tile is cut on the diagonal. The end rests upon the lower flange of the steel beam and is further supported by the lip resting upon the adjacent tile.

As the floorbeams are placed 2 feet apart, on centers, they may be very light, beams from 4 inches to 6 inches in depth being usually employed. Though a number of these small beams are required, the steel construction is not heavier than that in many of the other systems.

11. Among the points of superiority claimed for the Fawcett system of construction are the following: On account of the tubular shape of the tile, the amount of concrete required in the floor construction is greatly diminished. Owing to the circulation of air around the steel floorbeams,

TABLE V

WEIGHT OF MATERIALS USED IN THE FAWCETT FIREPROOF-FLOOR SYSTEM

Materials Used in Floors	Depth of Beams. Inches							
	4	5	6	7	8	9	10	12
	Weight of Floor Material per Square Foot of Surface. Pounds							
Steel beams . .	3.7	5.1	6.9	8.9	10.5	12.2	15.5	19.1
Lintels	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
Concrete	26.0	32.5	39.0	45.5	52.0	58.5	65.0	78.0
Wood floor . . .	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Plastering . . .	7.0	7.5	7.0	7.0	7.0	7.0	7.0	7.0
Total dead weight	55.2	63.1	71.4	79.9	88.0	96.2	106.0	122.6

an ample fire-protection is said to be provided, though recent tests by the New York Building Department do not demonstrate that this floor system is particularly adapted to withstand the effect of a severe fire and its attending destructive agent, water. A saving in depth of the floor construction is gained over many systems, the shallowest in the Fawcett system being 9 inches, while the deepest from the top of the finished floor to the finished ceiling is 11 inches. The shallowness of a floor construction can only be considered economical by reducing the height of a building where the girders are so arranged that they will come in line

In making the drop, or compact, test, three lintels, 9 inches wide by $11\frac{1}{4}$ inches deep, were set side by side between 10-inch I beams 5 feet from center to center. Upon them, a 2-inch bed of cement was spread. When a beam weighing 300 pounds was dropped a distance of 6 feet so as to strike endwise near the center of the section, two of the lintels were fractured by the blow, which amounted to an estimated impact of 10,000 pounds.

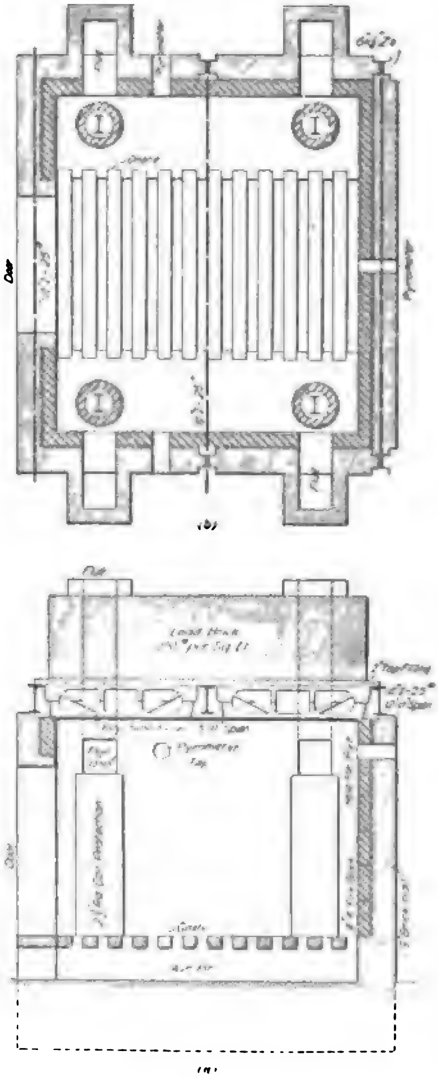


FIG. 9

The steel floorbeams are required to be placed about 5 feet from center to center, and, as will be observed from the above tests, this system is able to bear the usual loads encountered.

15. The refractory properties of the materials, or the fire-resisting properties of the system, have been given the test required by the Bureau of Building Inspection of Philadelphia. The test was made with a brick kiln, shown in Fig. 9, in which (*a*) is a vertical section and (*b*) a horizontal section. Its dimensions

were 8 feet 10½ inches by 10 feet inside and 6 feet 4 inches high from the top of the grate bars to the top of the ceiling. The grate surface was 5 feet by 10 feet and consisted of the regular boiler-furnace grate bars spaced 4 inches apart. The ash-pit under the grate bars was about 12 inches high. Four chimneys, extending 3 feet above the top of the floor covering and connected with the kiln by 13-inch openings in the wall located 6 inches below the ceiling, were placed in the corners of the kiln; three openings 6 inches in diameter and 6 inches below the ceiling were left in the rear and side walls for the insertion of the pyrometer, or instrument for measuring high temperatures. The kiln was covered with two bays of Keystone fireproof flooring carried by three 10-inch I beams, weighing 25 pounds per foot and spaced 5 feet apart. The floor was finished flush with the top of the beams and was 11½ inches thick; it was covered with 2 inches of concrete filling made of one part of cement and five parts of gravel. The floor was loaded with loose brick to a weight of 500 pounds per square foot distributed over 47½ square feet, and none of the weight was placed directly over the floorbeams. The walls were lined on three sides with regular Keystone fireproof blocks and on the other side with special blocks at least 4 inches thick. Four columns of 8-inch I beams 4 feet 9 inches long were placed inside the kiln opposite each chimney flue. These were covered with Keystone fireproof column protection 10 inches in diameter and 2½ inches thick; some of them were filled with a concrete composed of one part of cement and eight parts of gravel. The interior of the test kiln, walls, and ceilings, as well as the columns, was finished with a coat of Victor wall plaster ⅝ inch thick. The fire was started on the grates with wood for fuel and a temperature of 1,100° was shortly reached; this was increased in 3 hours to 2,300°. So intense was the heat that a ⅞-inch bar of iron melted during the test. At no time during the firing was there any appreciable warmth on the top of the floor or at the end of a floorbeam that projected beyond the kiln. Three hours after the fire had started, the door was opened and a stream of water from a

$1\frac{1}{2}$-inch hose having a $\frac{3}{4}$-inch nozzle, under a pressure of [illegible] pounds, was played upon the ceiling and columns until the fire was out. A careful examination then showed that the plastering had fallen to a considerable extent but the column protection and floor were intact, only needing replastering.

RAPP SYSTEM

16. The system of construction shown in Fig. 10 (*a*), (*b*), and (*c*) is known as the **Rapp system** of fireproof-floor construction. Its several types, while differing from each other in detail, are practically the same in their main features and are known to the manufacturers as types A, B, and C. It is put in place without the use of a temporary centering. Its distinguishing feature is the special arched T bars *a* sprung between the lower flanges of the steel floor-beams, as shown in Fig. 10 (*a*). These bars are formed of sheet metal about $\frac{1}{8}$ inch in thickness and bent, while cold, to the form shown in Fig. 10 (*d*). They are spaced about $8\frac{1}{4}$ inches from center to center, so that a brick will set neatly between the vertical legs of the T's, which are held in place during erection by the bar-iron separators *b*, *b*. When the T irons are in place, the bricks *c*, *c* are laid on edge and wedged with slate, the joints then being grouted. The bricks supported by the T bars form a permanent centering on which is deposited the concrete filling *d*; this concrete is composed of one part of Portland cement, in bulk, seven parts of cinders, and about one and one-half parts of sand. The sleepers *e* are laid perpendicular to the steel beams and filled between with a poor concrete filling. The usual rough flooring is spiked to the sleepers, while the finished floor is laid diagonally or at right angles to the under floor.

When a flat ceiling is required, either for appearance or for the protection of the lower flanges of the steel floorbeams, it is put in place on wire lathing secured to bar-iron supports *f* clamped to the lower flanges of the I beams, as shown at *f* in Fig. 10 (*a*). Where the brick filling between the T bars

laid with the bricks on edge, as shown in this figure, the construction complies with the rules and regulations of the Building Departments in several cities.

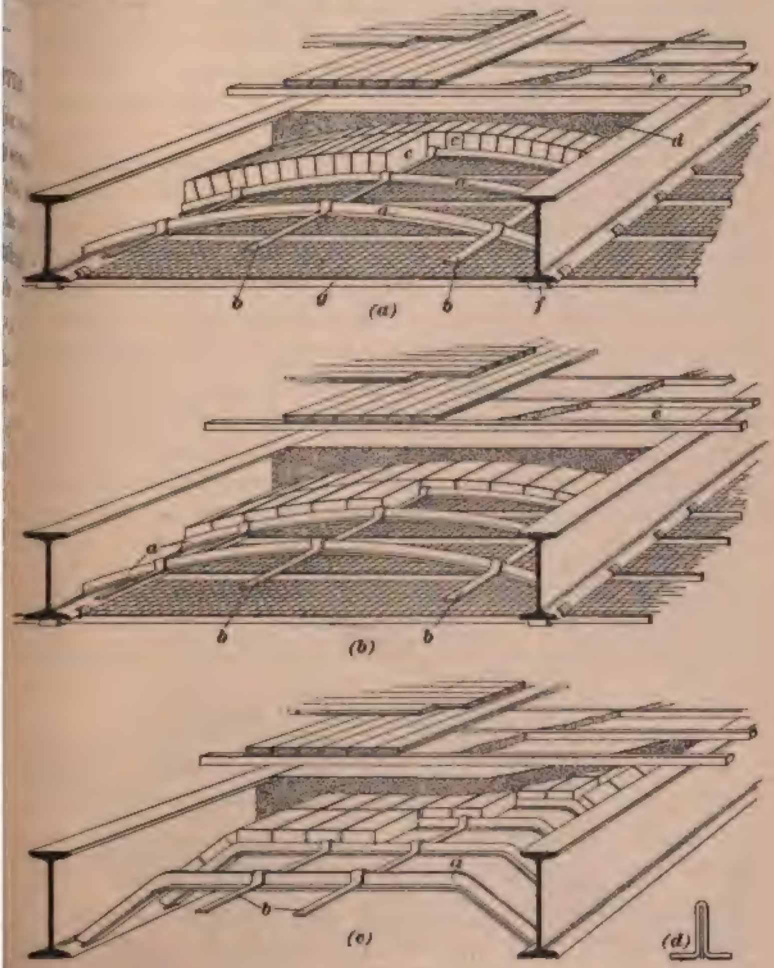


Fig. 10

In the construction shown in Fig. 10 (*b*), which is known to the manufacturers as type B, the bricks are laid flat. This makes a much lighter construction, but one that is fully as

strong as the type just described, since the principal strength of the floor is due to the monolithic slab of concrete reenforced by the **T** irons.

Where a paneled-ceiling effect is desired, the system known as type C, Fig. 10 (*c*), is adopted. The angle that the **T** irons make with the web of the steel beam may be made to suit the requirements of the decorative treatment beneath; in fact, they may be joined at any angle from vertical to 45°. The bricks are laid between the **T**'s, as in the construction shown in Fig. 10 (*a*) and (*b*), and the concrete filling is put in place in the same manner. This is the lightest construction of the three systems, but it possesses considerable strength. In the figure, the floor is shown without the flat ceiling, though this can be introduced if the additional protection is needed. During a test made by constructing an arch in this manner with 5 inches of concrete filling on the top of the brick, and having a load of 1,004 pounds per square foot, the deflection at the center of the arch was only $\frac{1}{2}$ inch and this was partially due to the compression of the concrete, as the floor had not been in position a sufficient length of time for it to thoroughly set.

17. Another type of construction known by this name is shown in Fig. 11. This construction is not arched, but the **T** irons lie flat and support the bricks by their transverse

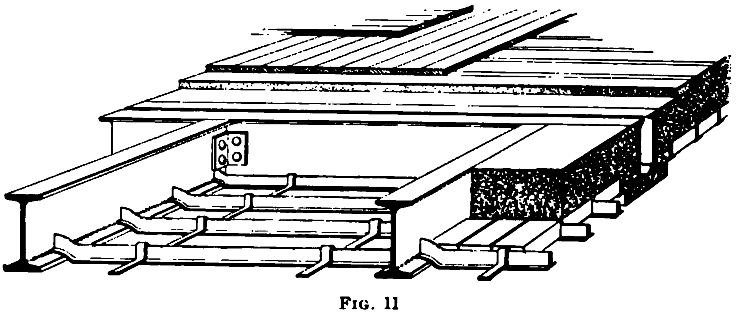


Fig. 11

strength. The concrete is tamped in place upon the bricks, as shown at *b*. The strength of the system depends on the transverse resistance of the slab of concrete and the **T** bars.

The details of this system of construction are the same as in the other types.

18. The Rapp system of fireproofing is adapted to many types of buildings, while the principle of supporting bricks, fireproof tile, or slabs of fireproofing material between **T** irons is applicable to many situations, and can be employed in the construction of roofs and domes. The floor is rapidly put in

TABLE VI

WEIGHT OF RAPP FIREPROOF-FLOOR SYSTEM

Type	Size of Beam Inches	Weight per Square Foot Pounds	Type	Size of Beam Inches	Weight per Square Foot Pounds
A	6	40.03	C	7	31.78
	7	42.53		8	32.78
	8	45.03		9	33.83
	9	47.53		10	35.83
	10	50.03		12	36.88
	12	55.03		15	39.98
	15	62.53	D	4	27.28
B	6	33.78		6	37.28
	7	35.03		7	42.28
	8	37.53		8	47.28
	9	40.03		9	52.28
	10	42.53		10	57.28
	12	47.53		12	67.28
	15	55.03		15	82.28

place, and owing to the fact that the simplest materials are employed, delays in construction are not liable to occur.

19. The approximate weights of the several types of the Rapp system are given in Table VI.

20. Notwithstanding the fact that the lower flanges of the steel beams and the metal **T** bars are protected by plaster, the system is not as efficiently fireproof as might be desired. It has been extensively used in hotel and

apartment-house construction on account
cost, but it can hardly be considered a tho
system, as the results of the following test,
under the supervision of the Department
New York City, will show. The method
test and its results are briefly stated as f
enclosure, provided with flues and grate ba
struction to that described in Art. 15, *Fir*
was used. The top of the kiln was di
panels, or bays, 4 feet wide, which was
the **T** bars; these were covered by the R
shown in Fig. 11. With wood as fuel, a
ging from 2,000° to 2,300° was maintained
the end of that time water was applied t
nozzle under a pressure of 60 pounds. Fo
directed against the ceiling, and for the n
was played against the walls and ceiling,
the latter. The water was then shut off
top of the floor for 5 minutes under a low
it entirely.

During the test, the floor was require
pounds per square foot, uniformly distri
any way bearing upon the beams. The m
when the fire was the hottest was equal
cooling, the deflection was reduced to
½ inch. An examination of the ceiling sho
were broken, cracked, or had fallen from th
from the east arch, and 2 from the west arch.
in the entire ceiling, 128, or about 17 per c
Some of the **T** iron bars had sagged con
plaster had fallen from most of the wire

STEEL-PLATE SYSTEMS

21. There are several systems of fireproof-floor construction that employ bent, crimped, or corrugated steel plate as the supporting medium between the steel beams. The plate, however, will not stand great heat, and unless these systems are protected beneath by a suspended ceiling, they are not satisfactory. Also, the moisture in the warm air of the room condenses on the steel plate to such an extent as to create dripping, which is likely to damage the goods beneath, besides corroding the plates.

22. The **corrugated-steel arch,** which is the simplest of these systems, is shown in Fig. 12. It consists of corrugated sheet iron of No. 16 gauge, bent or sprung between

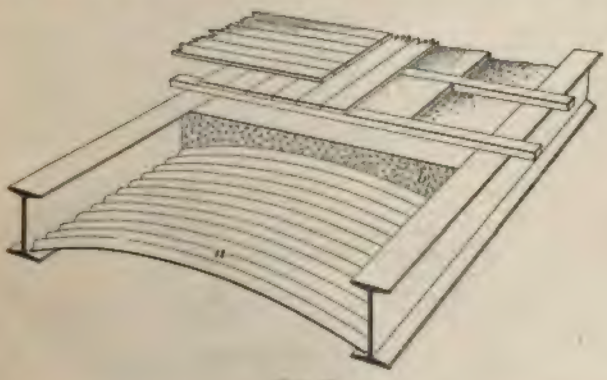


FIG. 12

the lower flanges of the steel floorbeams, as shown at *a*. The corrugated sheet bent in this way possesses great strength, and not only assists in supporting the floor load but acts as a center upon which the concrete *b* may be dumped and tamped. Upon the concrete, any finished floor may be laid.

Tie-rods between the steel beams are not necessary where the arches are placed side by side and where the thrusts counteract each other. However, they should be used to hold the beams from any side deflection during construction.

When tie-rods are used, they should lie in a corrugation
to enter the steel beams as near the lower flange as poss

23. The weights of this system of floor construction
different sizes of supporting beams are given in Table V

TABLE VII

WEIGHT OF CORRUGATED-IRON-AND-CONCRETE AR

Size of Beam Inches	Weight per Square Foot Pounds	Size of Beam Inches	Weight per Square Foot Pounds
6	26.5	10	36.5
7	29.0	12	41.5
8	31.5	15	49.0
9	34.0		

24. The **multiplex steel-plate floor** construction
shown in Fig. 13, consists of a plate *a*, bent as shown,
rests on the steel floorbeams and provides a mean

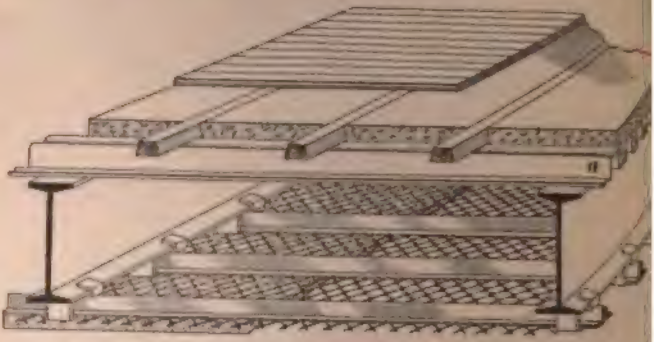


FIG. 13

supporting the concrete and the floor load. This
steel is usually galvanized to protect it from co
which is likely to occur in this type of constructio
ends of the plates are sometimes anchored to
flange of the steel I beams by driving a rod,

TABLE VIII

SAFE LOADS OF THE MULTIPLEX STEEL PLATE SYSTEM WHEN FILLED WITH CONCRETE 1 INCH ABOVE PLATE

Depth of Plate Inches	Metal Gauge Number	Dead Load per Square Foot Pounds			Distance Between Supports. Feet							
		Concrete	Plate	Total	3	4	5	6	7	8	9	10
					Load per Square Foot. Pounds							
4	16	14.1	7.9	22.0	4,230	2,370	1,510	1,040	760	575	450	360
4	18	14.1	6.3	20.4	3,180	1,780	1,130	780	570	430	335	270
4	20	14.1	4.7	18.8	2,260	1,265	800	550	400	300	235	185
4	22	14.1	3.9	18.0	1,810	1,010	640	435	320	240	185	145
4	24	14.1	3.2	17.3	1,408	792	507	352	258	198	156	127
3½	16	13.1	7.3	20.4	3.755	2,100	1,340	915	672	510	400	320
3½	18	13.1	5.8	18.9	2,815	1,575	1,000	790	510	380	295	235
3½	20	13.1	4.4	17.5	1,990	1,115	705	485	350	265	205	165
3½	22	13.1	3.6	16.7	1,500	840	530	360	265	201	150	120
3½	24	13.1	2.9	16.0	1,280	720	461	320	235	180	142	115
3	16	11.3	6.7	18.0	2,820	1,610	1,025	705	515	390	305	240
3	18	11.3	5.3	16.6	2,165	1,210	760	530	385	290	235	180
3	20	11.3	4.0	15.3	1,730	970	615	420	305	230	180	145
3	22	11.3	3.3	14.6	1,230	685	435	295	215	160	125	100
3	24	11.3	2.7	14.0	978	550	352	244	169	137	109	88
2½	16	9.8	6.0	15.8	2,260	1,265	905	555	405	305	240	190
2½	18	9.8	4.8	14.6	1,700	950	605	415	305	230	180	140
2½	20	9.8	3.6	13.4	1,210	675	430	295	215	160	125	100
2½	22	9.8	3.0	12.8	970	540	340	235	175	130	100	80
2½	24	9.8	2.4	12.2	770	433	277	192	143	108	86	69
2	16	7.5	5.4	12.9	1,330	745	475	325	235	180	140	110
2	18	7.5	4.3	11.8	1,005	560	355	245	180	135	120	85
2	20	7.5	3.3	10.8	810	450	285	200	145	110	85	65
2	22	7.5	2.7	10.2	640	350	230	155	115	85	65	50
2	24	7.5	2.2	9.7	454	255	163	117	87	65	52	41

in diameter, through the plates and turning it around the flange of the beams.

Where a flat ceiling is desired, it is suspended by the usual means and the plaster supported upon woven-wire lathing. In this construction, the lathing is carried upon special pressed-steel furring strips *b*.

25. The concrete used with the multiplex system consists of one part of Portland cement, three parts of sand, and five parts of furnace slag, composed of $\frac{1}{2}$-inch screenings; or, one part of Portland cement, three parts of sand, five parts of limestone or $\frac{1}{2}$-inch screenings of gravel. The first mixture weighs about 90 pounds per cubic foot while the latter weighs 120 to 140 pounds. This floor construction is comparatively light in weight, as will be seen from Table VIII, which gives the total dead load for the concrete and steel plate, and the itemized loads for different spans and different thicknesses of steel plate.

The strength of this system of flooring, also designated in Table VIII, is considerable, since the concrete and steel troughs apparently reenforce each other; the safe loads given in the table have been obtained from actual tests.

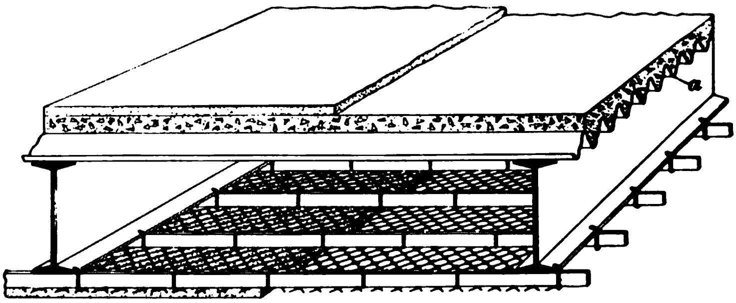


FIG. 14

26. The **Buckeye fireproof flooring** is very similar to the construction just described. It consists of a metallic trough-shaped section *a*, shown in Fig. 14, that supports a layer of concrete. The ceiling, which is necessary to protect

beams and lintels, is laid on expanded-metal lath sup-ted by bars of iron that are secured, by clips, to the lower ge of the floorbeams, as shown in the figure. A dimen-ed section of the tallic trough is wn in Fig. 15. e concrete used is erally composed one part of Port-d cement, three ts of sand, and five parts of broken stone. In addition its acting as a reenforcement for the trough, it also pro-ts the metal from corrosion.

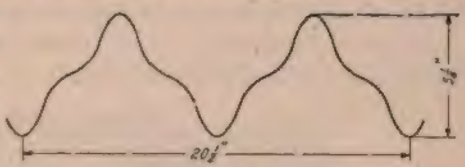


FIG. 15

27. The floor, when complete, will weigh about 35 pounds r square foot; the safe load that it will carry, adopting a tor of safety of 4, may be ascertained from Table IX.

TABLE IX

SAFE LOADS FOR BUCKEYE FIREPROOF FLOOR

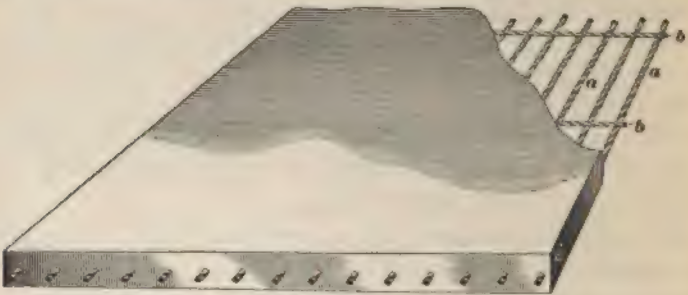
Troughs 5¼ Inches Deep, 10¼ Inches Wide, With a ½-Inch Cement Wearing Surface

Span		No. 18 Gauge Pounds	No. 20 Gauge Pounds	No. 22 Gauge Pounds	No. 24 Gauge Pounds
Feet	Inches				
3	0	1,050	800	580	450
3	6	820	570	425	320
4	0	675	425	315	230
4	6	570	320	240	190
5	0	475	250	180	135
5	6	420	200	140	

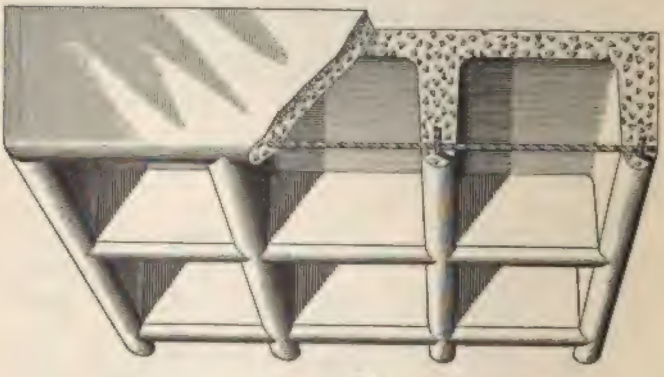
RANSOME AND SIMILAR SYSTEMS

28. The **Ransome system** of floor construction consists in placing twisted steel or iron bars in the concrete slab forming the basis of the floor system. It is a development of the earliest form of reenforced concrete construction, which was known as the Hyatt system and consisted of a concrete

floor reenforced with plates of iron set on edge, that is, placed vertically and pierced with openings through which round connecting-rods were passed. This network of metal formed a kind of gridiron and was placed at the bottom of the concrete slab so that the desired tensile strength might be obtained. In the Ransome system, no attempt is made to tie the twisted



(a)



(b)

FIG. 16

bars together, though sometimes they are laid across each other, as shown in Fig. 16 (*a*). Here the bars *a*, *a* are the main tension bars while *b*, *b* are auxiliary. It is stronger than the old Hyatt system and is somewhat cheaper. Owing to the twist in the rods they are securely embedded in the concrete so that they cannot slip, and, hence, they are efficient

in resisting the tensile stress existing at the under side of the slab and transferred to them by the concrete.

The Ransome system of flooring has been used for clear spans as great as 35 feet and offers considerable resistance to transverse stress. The results of a careful test showed that a floor 15 feet by 22 feet, built on this system, supported without failure or excessive deflection a load of 400 pounds per square foot for a month.

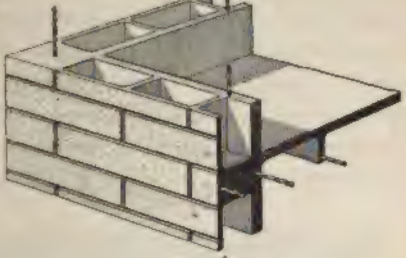


FIG. 17

When a paneled-ceiling effect is desired, the Ransome construction may be readily used and is commonly reenforced as shown in Fig. 16 (*b*). The system has admirable fire- and water-resisting qualities since none of the metallic work is exposed. The hollow wall construction of the Ransome system is shown in Fig. 17.

29. The **corrugated-bar system** is a product of the Expanded Metal Fireproofing Company. Its principles of construction are precisely the same as those employed in the Ransome system, namely, the reenforcement of the monolithic floor by the use of tension bars or rods. The difference between the two systems is in the tension bars, which in this


FIG. 18

system have projections on the four sides at intervals, as shown in Fig. 18. By roughening the bar in this manner, there is never any danger of diminution of the strength of the system by the destruction of the adhesion between the concrete and the metal. Both the Ransome and the corrugated-bar systems depend, for their efficiency in resisting

transverse stress, on the united action and perfect union of the concrete and the reenforcing steel bars, and undoubtedly by twisting or roughening the bars some gain in the transverse resistance of the floor system is attained.

The corrugated-bar system is shown in Fig. 19, where *a*, *a* are the auxiliary floorbeams and *b*, *b* the reenforcing corrugated bars. These corrugated bars are about ½ inch square and are bent upwards at both ends, so as to hook around the upper flange of the steel floorbeams, as shown at *c*. The greatest efficiency of this system of construction is for spans from 8 to 15 feet in length. For spans over 15 feet in length, the dead load is so increased, owing to the thickness of the concrete required, that this system no longer remains

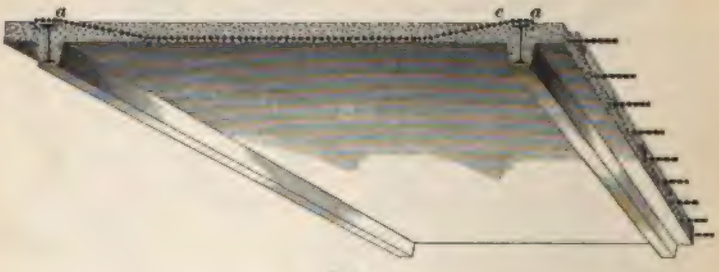


FIG. 19

economical. The thickness of the concrete ranges from 2 inches for spans of 4 feet to 7½ inches for spans of 15 feet.

30. The strength of the corrugated-bar system of floor construction is sufficient for the usual spans, as will be observed from Table X, which gives the breaking load for this system of floor construction. By allowing a factor of safety of from 4 to 6, the safe live load may readily be determined.

31. The **Metropolitan system** of fireproof-floor construction is shown in Fig. 20 (*a*), (*b*), and (*c*). In laying, the floor cables, made by twisting together two strands of No. 12 gauge galvanized-iron wire, are secured to the upper flanges of the auxiliary floorbeams at intervals of 1½ inches, as shown in Fig. 20 (*a*), in which *a* are the galvanized wire

TABLE X

BREAKING LOADS OF CORRUGATED-STEEL BAR FLOOR SYSTEM

Type of Floor		Thickness of Slab Inches	Spacing of Bars Inches	Span in Feet: 8		9		10		11		12		13		14		15		Moment of Resistance for Concentrated Loads
				Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	Uniform Load. Pounds per Square Foot	Concentrated Load Tons	
½-Inch Square Corrugated Steel Bars Spaced So That the Slabs Are of Equal Strength in Tension and Compression	Cinder Concrete	3½	24	317	.51															24,400
		4	20½	445	.71	342	.63													34,200
		4½	18	590	.95	454	.84	363	.75											45,400
		5	15½	760	1.22	584	1.08	467	.97	380	.88	315	.81							58,400
		5½	14	950	1.53	732	1.36	585	1.22	475	1.11	396	1.02	334	.94					73,200
		6	12½	1,160	1.86	894	1.65	715	1.49	580	1.35	485	1.24	407	1.14	348	1.07	300	1.00	89,400
		6½	11½	1,400	2.24	1,074	1.98	860	1.78	700	1.62	581	1.50	490	1.37	417	1.28	360	1.19	107,400
		7	10½	1,640	2.64	1,266	2.35	1,010	2.10	820	1.91	682	1.76	585	1.62	492	1.51	425	1.41	126,600
		7½	10	1,920	3.08	1,480	2.74	1,170	2.46	960	2.24	800	2.06	673	1.80	575	1.76	495	1.64	148,000
	Rock Concrete	3½	18½	450	.72	347	.64													34,700
		4	15½	630	1.01	486	.90	388	.80	316	.73									48,600
		4½	13½	836	1.34	644	1.19	515	1.07	418	.97	347	.90							64,400
		5	12	1,080	1.73	830	1.54	665	1.38	540	1.25	450	1.15	378	1.06	323	.99			83,000
		5½	10½	1,350	2.16	1,034	1.91	825	1.72	670	1.56	560	1.43	470	1.32	400	1.23	345	1.14	103,400
		6	9½	1,650	2.64	1,268	2.35	1,020	2.11	825	1.92	685	1.76	580	1.62	495	1.51	425	1.41	126,800
		6½	8½	1,980	3.17	1,520	2.82	1,220	2.52	988	2.30	820	2.11	690	1.94	590	1.81	510	1.69	152,000
		7	8	2,330	3.75	1,798	3.33	1,440	2.98	1,170	2.71	970	2.50	820	2.30	700	2.14	600	2.00	179,800
		7½	7½	2,720	4.37	2,094	3.87	1,680	3.47	1,360	3.15	1,130	2.91	950	2.68	815	2.50	700	2.33	209,400

cables; *c*, the hooks by which they are secured to the upper flange; and *d*, is a piece of bar or pipe laid centrally between the I beams so as to stretch the cables and give them the camber, as shown in (*b*) and (*c*). After the cables are in place, a composition of one part of wood shavings and five parts of plaster of Paris, mixed to a thin paste with water,

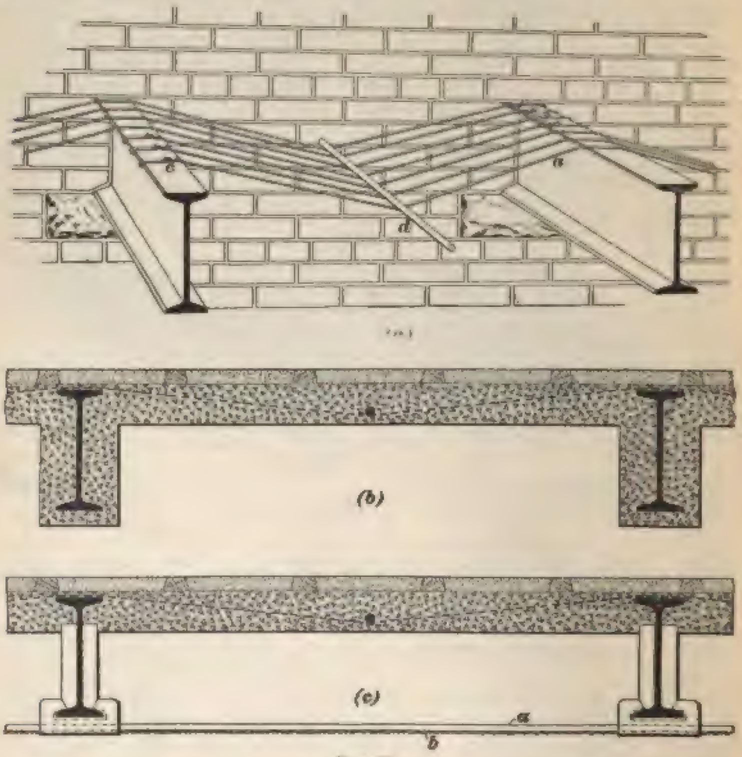


Fig. 20

is poured in place upon the temporary wooden centerings. The centering is placed 1 inch below the round iron bars, or pipes, used at the center of the span to stretch the cable. The composition, which hardens quickly, is brought to a level about ½ inch above the level of the floorbeams and when finished, forms a reenforced slab about 4½ inches thick, which is ready for the laying of the sleepers or any finished

floor on the top and the plastering beneath. The auxiliary beams, as well as the principal girders, are protected by the same material, which is usually cast in place about the webs and lower flanges, always being so arranged that 2 inches will be the minimum thickness of the fireproofing.

In this system no tie-rods are required, as the floor loads are transmitted by a suspension system rather than an arch. The tendency to overturn or twist the I beams is counteracted by the cables in the adjacent panels, while in the end panel the beam can be tied to the walls or structure, or it can be made sufficiently heavy to resist any tendency of this kind. This tendency is also partially eliminated by the fireproofing material, which acts as a compression member between the upper flanges of the steel beams. In (*b*) is shown the completed flooring with a paneled effect beneath. When a flat ceiling is desired the construction shown in (*c*) is usually employed. Iron bars are placed upon or suspended from the lower flanges of the floorbeams and upon them is wired herring-bone, pressed-steel lathing that has been dipped in asphaltum. Upon the lathing, the usual plastered ceiling is applied. In the figure, *a* is one of the supporting bars and *b* the metallic lath.

32. The distinguishing features of the Metropolitan system are the utilization of plaster of Paris or a mixture of plaster of Paris and wood shavings instead of stone or cinder concrete, and the adoption of wire cables instead of iron or steel rods or bars. The principal point of superiority of this system is its extreme lightness; its average weight, without considering the steel floorbeams, is from 18 to 20 pounds, but where a plastered ceiling is used the weight is increased from 6 to 8 pounds. The transverse strength of the system is ample for any usual floor loads, as the actual tests given in Table XI will demonstrate.

TABLE XI

RESULTS OF TESTS FOR STRENGTH OF METROPOLITAN FLOOR SYSTEM

Distance Between Centers of Beams		Length of Section Tested		Area Tested	Total Load Applied	Load per Square Foot	Remarks
Feet	Inches	Feet	Inches	Square Feet	Pounds	Pounds	
6	10	2	0	10.832	5,395	498	Did not fail.
4	7	1	0	4.166	5,630	1,351	Did not fail.
5	5	0	9½	3.958	7,600	1,920	Two cables broke on one side only.
5	0	1	6	6.844	8,930	1,304	Broke on both sides.
5	6	5	6	12.605	15,146	1,202	Not tested to destruction; adjoining section commenced to lift.
4	6	2	6	10.105	15,682	1,551	Failed by the breaking of some of the cables on one side.
3	9	2	6	8.229	18,950	2,302	Failed by adjoining sections lifting.
5	6	5	¼	25.31	18,422	728	Failed by beam bending, hooks straightening out, and some breaking.
4	6	5	½	20.38	29,314	1,438	Failed by adjoining section lifting.
5	0	2	0	9.125	10,882	1,192	Separate wires on each section fastened on outside beams with hooks, and on center beam with a wire around the bottom of beam connecting the wires. Adjoining section lifted about ¼ in.
5	1¼	2	0	9.333	9,775	1,047	Failed by outside beams tilting and drawing in about ½ inch toward the center until outside arches lifted. No tie-rods used.
7	0	2	6¼	16.59	17,210	1,037	Failed by deflection and adjoining arches lifting. No wires broken.
8	0	2	6⅛	18.93	16,245	858	Failed by deflection and adjoining arches lifting. No wires broken.

TABLE XI—(Continued)

Distance Between Centers of Beams		Length of Section Tested		Area Tested	Total Load Applied	Load per Square Foot	Remarks
Feet	Inches	Feet	Inches	Square Feet	Pounds	Pounds	
7	0	2	6¼	16.48	18,151	1,101	Failed by deflection and adjoining arches lifting. No wires broken.
6	0	2	6⅛	14.00	18,891	1,350	Failed by deflection and adjoining arches lifting. No wires broken, but outside beams bent about 1 inch.
5	6	2	6	12.68	14,076	1,110	Failed by deflection and adjoining arches lifting. No wires broken.
5	6	2	6⅛	12.71	16,526	1,300	12 wires 2½ inches apart. Failed by all the wires breaking close to beam on one side.
7	0	2	6½	16.64	17,660	1,061	Heavy rain storm; arch not protected. Tested following day. Failed by deflection and adjoining arches lifting. No wires broken.
8	0	2	6	18.88	16,265	861	Failed by deflection and adjoining arches lifting.
6	0	2	6½	14.08	18,710	1,328	Failed by deflection and adjoining arches lifting.
5	6	2	6¾	12.87	17,845	1,386	Failed by deflection and adjoining arches lifting. These arches were built between 15-inch beams, without beam protection.
		2	0	10.667	5,923	555	Test made in Baker Building, Philadelphia, Pa., and was part of permanent floor. Not tested to destruction.
5	10½	2	0	11.00	8,782	798	Test made in Baker Building, Philadelphia, Pa., and was part of permanent floor. Not tested to destruction.
5	6	2	8½	13.74	11,116	809	Not tested to destruction.
5	6	2	6	12.63	9,510	753	Not tested to destruction.

FIREPROOFING FOR COLUMNS

33. Since the columns in a building are the most important structural members, too much care and attention cannot be given to their proper fireproofing. The stability of the entire structure depends on them, for the failure of one column in the basement would be likely to hasten the destruction of the building. It is not uncommon for a large percentage of the entire weight of the building to be carried on one structural steel column, as where it supports a heavy trussed girder, which is often employed to span an entire building over an exchange or ballroom and must necessarily carry all of the floors above, often ten or more in number. An example of this kind exists in the Hotel Waldorf-Astoria, New York City, where a column sustains a load approximating 5,400,000 pounds.

The fireproofing around columns should be designed and constructed with the care and skill commensurate with their importance, and where the column is the main support of many stories, as occurs in the modern office building, the fireproofing material should be chosen and put in place with such care that any likelihood of its proving inefficient in case of fire is precluded. The several companies that construct fireproof-floor systems, provide systems using practically the same materials for the columns. However, because a floor system may be efficient, it is no proof that the same materials and methods will make an efficient fireproof covering for columns, and the most efficient system for column fireproofing should be used without reference to the system adopted for the floor construction.

34. The materials used in fireproofing columns are dense and porous terra-cotta, hollow clay tile; stone and cinder concrete; and expanded-metal and wire lathing covered with cement and fireproofing plasters. While the materials

used are comparatively few, the forms adopted and the methods employed are numerous. Many of the systems are far from efficient and their principal qualification seems to be cheapness of construction. Only the most efficient column fireproofing should be employed, for inferior methods are the source of much danger owing to the fact that they cover the metallic framework from inspection and hence serious corrosion is likely to take place unobserved.

The perfect column protection should have the following qualifications: It should be non-combustible, a non-conductor of heat, and refractory; impervious to moisture, though not necessarily dense, for the fact that it is porous does not always signify its permeability. It must resist fracture from sudden cooling and destruction from blows and streams of water. All joints must be resist moisture and fire. Air spaces provided in the material and between the material and the column offer additional resistance to fire, but they must be stopped at each floor and at several points between floors so as to prevent the possibility of a vertical flue being formed, which would prove destructive in case of fire.

35. The simplest forms of **terra-cotta column fireproofing** are shown in Fig. 21 (*a*) and (*b*); (*a*) shows a covering of terra-cotta segmental blocks protecting a cast-iron column. The inside faces of the blocks are provided

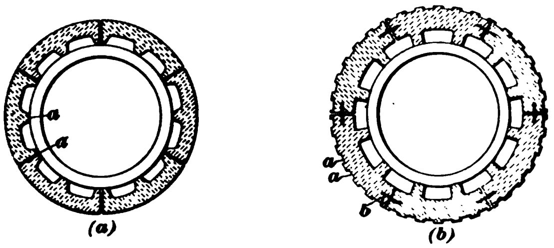


FIG. 21

with projections *a* that bear against the column and provide the air spaces between them, also a support for the blocks, thus preventing them from being displaced. In (*b*) is shown a very similar system provided with vertical grooves *a* on

the outside of the terra-cotta blocks in order to form a satisfactory base for the plaster. These blocks are set in place with cement or mortar and are held at the joints by the metal binders or keys *b*. The blocks are so arranged that all vertical joints will be broken, as shown in Fig. 22. The principal objection to this system is that the blocks are thin and do not adequately protect the columns from heat. The fireproofing is apt to be broken or displaced in case of fire and the joints are likely, in time, to be affected by moisture and heat.

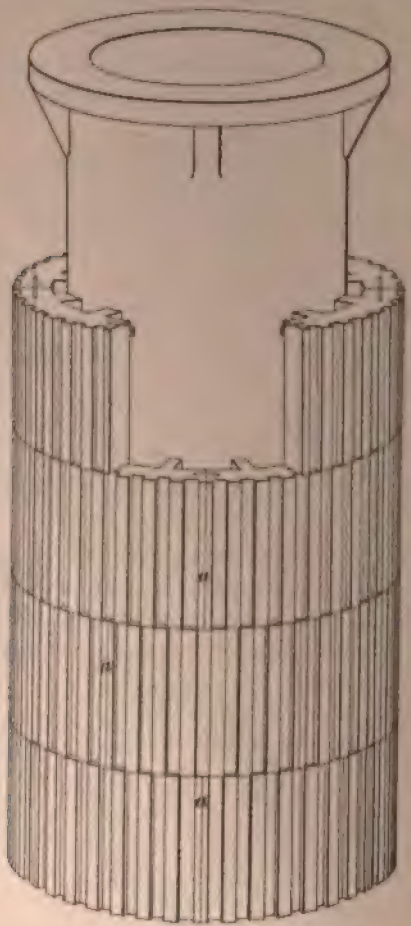


FIG. 22

To obviate these difficulties, the fireproofing tiles shown in Fig. 23 (*a*) and (*b*) may be used. (*a*) shows a method of arranging a joint that will not be likely to open, while additional fire-protection is provided by the use of the air spaces at *a*, *a*. The protection for the *Phœnix column* is shown in (*b*). The corners *a* of these blocks are molded to suit the requirements of the column; air spaces are provided, and the blocks may be bound at the joints with metal binders *b*. In both types of construction, the fireproofing is held firmly in place and protected against external pressure

by occasional header or thicker courses, which extend inwards and come in contact with the surface of the column.

36. The usual systems of fireproofing structural steel columns are shown in Fig. 24 (*a*) and (*b*). The terra-cotta blocks, shown in (*a*), are usually made about 3 inches thick

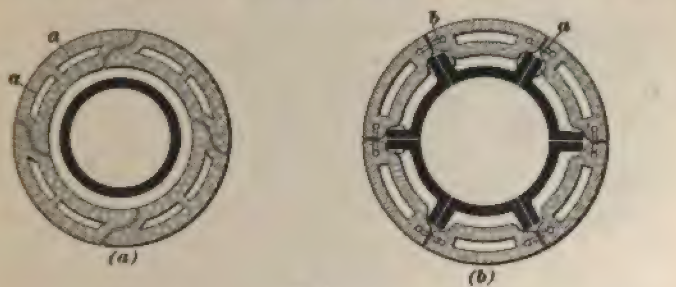


FIG. 23

and are provided with air spaces, as shown. They are usually backed with rough brickwork filling or poor concrete; in fact, this is the best construction, for without such a backing the fireproofing is not sufficiently strong to remain in place during a serious conflagration. The construction shown in

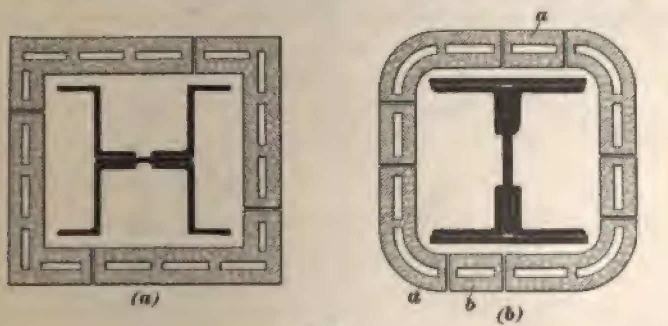


FIG. 24

(*b*) is similar to (*a*), with the exception that the corner blocks are rounded; by this method corner blocks may be made one size and by changing the length of the block *a*, the fireproofing may be made suitable for any column. Besides setting the terra-cotta tiles in cement mortar, or in

the o… be fireproof than
facto… in place by binding
with… a foot or so.

…cotta fireproofing for
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… diameter usually equal
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…per distance from the fill-
…ile or by projecting ribs …
…xterior tiles.

… **system** of column firepro…
…and the column sheets of
…e usual plaster is placed
…etween the column and the
…ring. This is accomplished

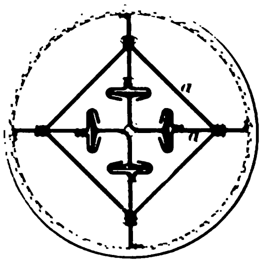


FIG. 27

… bent pieces *a* of sheet
… the column.

… it is desirable, for architec-
… the system of
… used. The …

is built to the required diameter by means of a light bar-iron frame and clamps *a*, *a*. The expanded metal is wrapped around the outside of this framework and plastered, as shown. Where the column is an important member in the structure, at least two coverings of expanded metal and plastering should be provided.

38. The arrangement of the fireproofing material when two coverings are required is shown in Fig. 28, which shows the section of a *Grey column* that was first protected by a wrapping of expanded metal and plaster, then the vertical metallic furring strips *a* were fastened in place, and upon them was secured a second wrapping of expanded metal, which carries the external plaster. In this manner, not only is an air space provided around the column, but a double covering is secured, so that in case the outside is damaged, the column is protected to a certain extent. This method is considered good practice and provides adequate protection, for the expanded metal and plaster forms a tenacious covering. If the spaces *b*, *b*, Fig. 28, were filled in with concrete and a coating of some waterproof material were placed on the outside of the plaster *c*, an almost indestructible type of column fireproofing would be obtained; this method should be employed where the column is of vital importance to the structure.

FIG. 28

39. The **Roebling system**, which is an excellent method of fireproofing columns, consists of woven-wire lathing and cinder concrete constructed as shown in Fig. 29. The woven wire, which contains the reenforcing wires every $7\frac{1}{2}$ inches, is wrapped around small channels or $\frac{5}{16}$-inch round rods *a*, *a*, which form the furring that is held 2 or 3 inches from the columns by means of bar-iron supports, or clamps *b*. The space between the woven wire and the column is filled with cinder concrete of the usual proportions, while on the outside of the plaster coating the woven-wire lathing the finished plaster coating is put in place.

40. Concrete is used extensively for fireproofing col-
umns. Owing to its refractory properties and to the fact

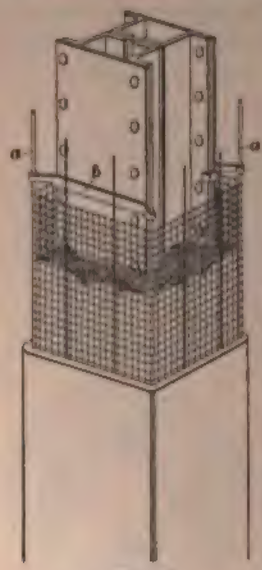


FIG. 29

that it can be readily molded to any
shape of column, and also because
the materials of which it is composed
can readily be obtained in any locality
at moderate expense, it makes a practical system of fireproofing that is
superior to the others. In Fig. 30
(*a*) and (*b*) is shown the section of a
Phœnix column completely protected
by a concrete casing. In (*a*) is shown
the mold around the concrete, which
is composed of 2-inch planks or
staves *c* held together with heavy bar
iron straps. These straps are made
in halves, the same as the molds, and
are hinged at *d*. They are bent to
form a clamp *a* and are provided with
bolts so that the mold can be drawn
tightly together. These molds are
made in about 4-foot lengths, and when the concrete has
been tamped in place and has had time to set, the bolts are

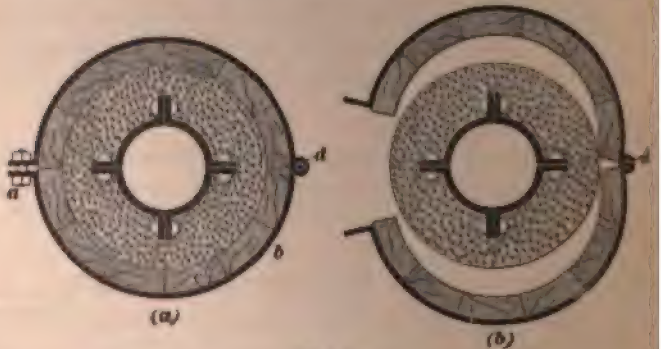


FIG. 30

removed and the molds taken away, as shown in (*b*),
applied to another column or another section of the lengt

the same column, being supported in the proper position by a false work or scantling. By the use of a mold for each column, the fireproofing of all the columns can be carried along simultaneously. Since, however, 24 hours should elapse before the mold is removed, the columns should be fireproofed in sets of three or four, so that each set can be laid and tamped consecutively. The workmen may then return to the first group, in which the concrete has obtained the initial set, when they have completed the last group or set.

41. In Fig. 31 is shown a *box column* protected with concrete; this method can be used to advantage in ware-

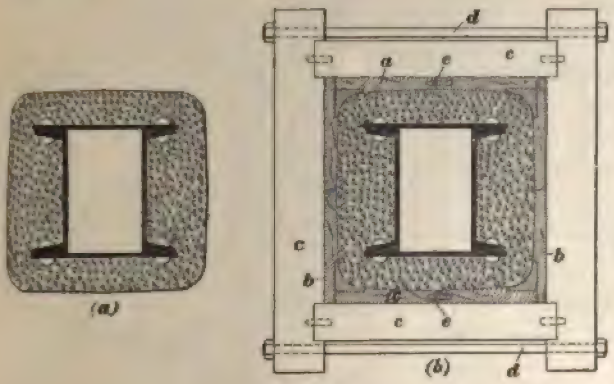


Fig. 31

houses. A section of the column, with its fireproof covering, is shown in (*a*). The mold, or form, for the concrete protection, shown in (*b*), is made in halves that are provided with dowel-pins *e*, *e*, so that the two sections will always come together evenly. The side pieces *a*, *a* of the mold are tapered or provided with a draft so that the form will readily draw away from the concrete when it has set. In order to make the column symmetrical, the other sides *b*, *b* of the mold are tapered toward the center. The wooden mold is held together by clamps *c*, *c* composed of 3″ × 4″ timbers and provided with bolts *d*. When the concrete has set, the

bolts are withdrawn and the mold removed from the column and set up at another place. The molds are made from 4 to 6 feet in length and are provided with a yoke or clamp at each end.

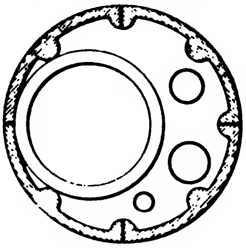
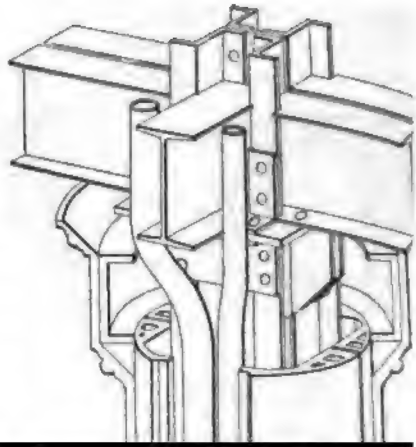


Fig. 32

The advantages of this system are obvious, for the concrete cannot readily be damaged or broken; it is an excellent non-conductor and protects the column not only from heat but from corrosion. When the location warrants it and the column is a closed section, as shown in Figs. 30 and 31, corrosion on the inside may be prevented by filling the column with concrete. The concrete is likely, also, to increase the strength of the column, and tends to promote rigidity throughout the structure. In fact, where the concrete is well tamped and is used inside of the column as well as outside, the



42. Supply and vent pipes should never be carried inside the fireproof casing and immediately adjacent to the columns, because the condensation on the pipes is likely to cause corrosion, and also because it is impossible to inspect or repair them without practically rebuilding the fireproofing. It has been a common practice, though one that cannot be recommended, to run all piping and electric conduits adjacent to the columns, and in some cases even the plinth plates between the columns of the several stories have been cut to provide a passage. Such a practice cannot be too severely criticized. Fig. 32 shows a cast-iron column provided with an eccentric fireproof covering so as to allow a passageway for pipes. This construction is objectionable from the fact that the pipes are not accessible for repairs and also because corrosion is likely to be effected in the columns through leakage or condensation on the pipes. In Fig. 33, the fireproofing around the columns is provided with a section that is readily removed. This obviates the necessity of destroying the fireproof covering in order to have access to the pipes or wiring that runs adjacent to the column. In removing this covering, however, it is necessary to destroy the plastering adjacent to it, and the work must be replastered. It does not, however, separate the piping from close proximity to the column as does the construction shown in Fig. 34, which is the method that should be used where it is absolutely necessary to run the pipes and conduits along the columns. The column is fireproofed independently of the pipes, which can be replaced or removed without injury to the column fireproofing, besides the possibility of condensation or leakage penetrating to the column is remote.

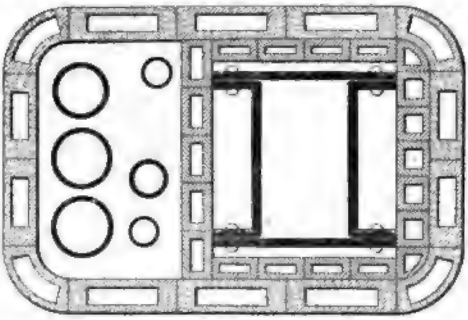


FIG. 34

FIREPROOF PARTITIONS

43. **Fireproof partitions** are constructed of either brick, hollow terra-cotta tile, or metallic lath and plaster. Brick partitions are too heavy for office-building work, and as they must necessarily be from 8 inches to a foot thick, they take up too much floor space. They also offer the objection that they cannot readily be removed to suit the requirements of tenants. The use of hollow terra-cotta tile obviates, to a

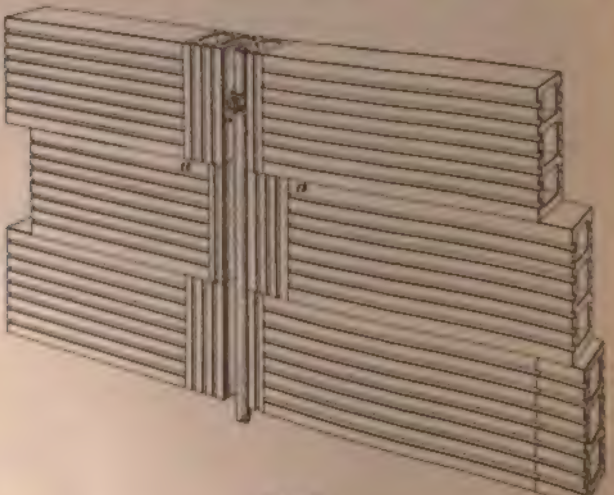


Fig. 35

great extent, all of these difficulties, and when properly laid they provide a light and refractory partition that may be located at any position on the floor and is readily removed.

44. The usual **terra-cotta tile partition** is shown in Fig. 35. Pipe chases are readily provided by the use of recessed hollow tiles *a*, *a*. The other tiles are also hollow, but the spaces usually run horizontally. The blocks are commonly set in lime mortar containing about 25 per cent

of cement, and are grooved upon the face so as to provide a key for plastering and avoid the necessity of wooden furring and lathing, or metallic lathing. Some quick-setting plaster is generally used for finishing the faces of the tile.

The type of hollow partition tile shown in Fig. 36 is known as the *Phœnix*. It is about 4 inches thick and 12 inches by 18 inches on the face, being hollow and provided with the dovetail grooves for plastering, as shown. Its lateral stiffness is greatly increased by the use of flat bar iron, which fits snugly in grooves *a* molded along the edges of the blocks.

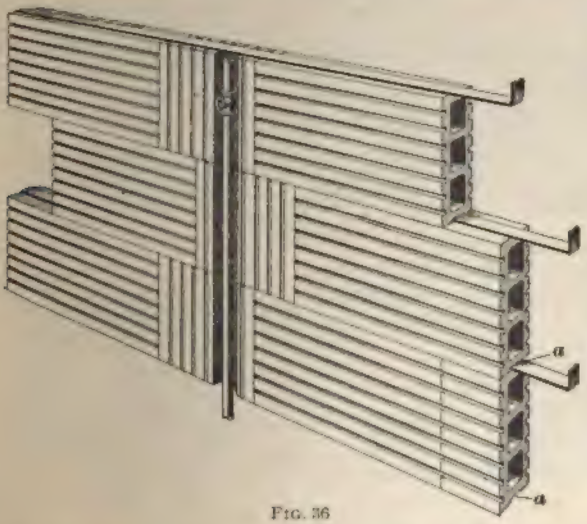


FIG. 36

Where these strengthening bars are used and the partition blocks are molded 4 inches in thickness, they have been successfully used for the outside walls of smelting works and refineries. In such positions, they have furnished the strength and resistance of a 12-inch brick wall at considerably less cost.

45. In building terra-cotta tile or any other fireproof partition, the practice of setting them on a wooden sill cannot be too strongly condemned. This method of construction has led to considerable fire-losses from the burning away of the sill and the ultimate destruction of the partition. The

best practice specifies that all fireproof partitions shall rest directly upon the concrete of the floor system.

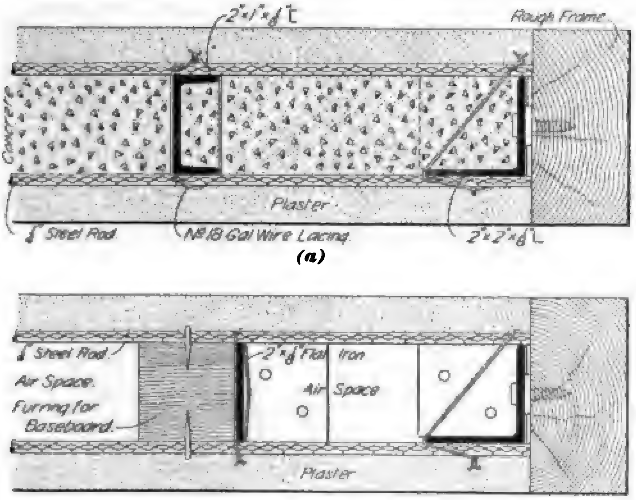
The hollow clay tile partition has no particular advantages over other fireproof partitions, but, unlike the ordinary lath-and-plaster partition or the brick wall, it combines considerable strength with lightness, and it is also entirely vermin-proof. The partitions, being hollow, do not readily transmit sound, and being made of a porous, non-conductive material

TABLE XII

WEIGHTS OF TERRA-COTTA PARTITIONS, FURRING, CEILING, AND ROOFING

	Thickness Inches	Weight per Square Foot Pounds
Hollow-brick partitions	3	15
Hollow-brick partitions	4	20
Hollow-brick partitions	5	24
Hollow-brick partitions	6	28
Porous terra-cotta partitions .	3	14
Porous terra-cotta partitions .	4	18
Porous terra-cotta partitions .	5	23
Porous terra-cotta partitions .	6	27
Hollow-brick furring	2	12
Porous terra-cotta furring . . .	2	8
Porous terra-cotta ceiling . . .	2	12
Porous terra-cotta ceiling . . .	3	15
Porous terra-cotta ceiling . . .	4	20

47. The **Roebling partitions** have all the qualifications of a good fireproof partition. They are somewhat stronger and more elastic than a hollow-tile or block partition of the same thickness, because of their monolithic construction and the consequent absence of any joints. They also have the advantage that pipes and conduits can be placed in any convenient location, while in the hollow terra-cotta tile partitions special recessed blocks must be provided or a channel gouged or broken in the tiles, thus seriously affecting their



(b)

Fig. 37

strength. The Roebling system provides either a solid or a hollow partition, as may be desired. The construction of the solid partition is shown in Fig. 37 (*a*), while the construction of the hollow partition is shown in (*b*). In both systems the Roebling wire lathing, with a solid steel stiffening rib woven in every $7\frac{1}{2}$ inches, is used for lathing and reenforcing the partition.

In the solid partition, the studding is composed of $2'' \times 1'' \times \frac{1}{8}''$ channels, which are turned on the end so that they may be nailed or spiked to the floor system above and

galvanized wire. The solid partition has
concrete 2 inches thick, as shown in the fig
faces are plastered in the usual manner.
only two coats of plaster are required—a b
ing coat. Its weight, including the plaster
32 pounds per square foot.

As the concrete will receive nails, no
necessary in order to attach the baseboa

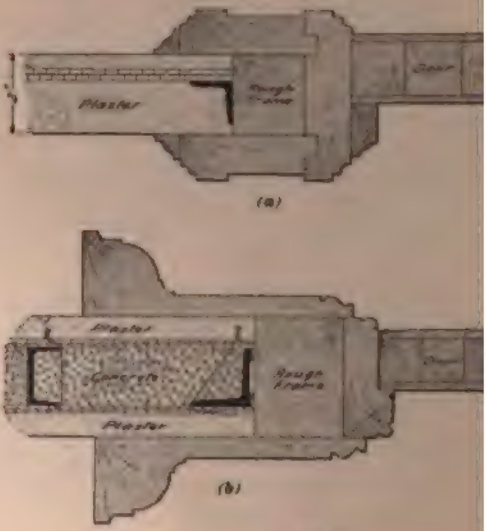


FIG. 28

picture mold. The size of the door fram
arrangement of the wood trim are show
and (*b*). It will be noticed that the partit

the solid partition in being lighter, for it is provided with an air space of about $1\frac{3}{4}$ inches between the plastered surfaces. This air space, besides providing an insulation against sound and heat, is also convenient for the placing of conduits for pipes or electric wires. In this method of constructing a partition, it is necessary to provide wood furring strips or blocks for the baseboard, chair rail, and picture mold. These are secured inside of the wire netting by staples and, wherever possible, should be composed of wood that has been subjected to an efficient process of fireproofing, as explained in Arts. **51** and **52**. The partition requires three-coat plaster work; that is, a scratch coat, brown coat, and finished coat, and weighs, including the plaster, about 22 pounds per square foot.

Of the types shown in Fig. 37, the solid partition offers the better fire-protection; the hollow partition is not adequate unless fireproof wood is used in its construction, for the falling or breaking off of the plaster coating and the woven wire will expose the wooden furring strips, which will furnish fuel for the fire.

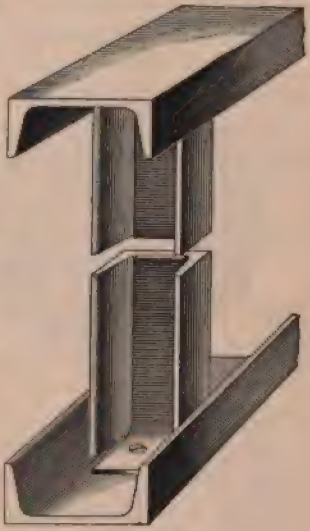


FIG. 39

49. The door frames and trim in all fireproof partitions, in first-class work, should be made of fireproof wood or covered with a metallic casing. Art metal work has become so practical and artistic as to provide thoroughly architectural and substantial casings for doors and frames, and also for baseboards and other finish.

50. Expanded-metal partitions are very similar, in construction, to the Roebling, the only difference being that expanded metal is used for the lathing instead of woven wire. For the studding in this system, 2-inch, 3-inch, and

4-inch steel channels are placed on 16-inch centers ar braced once in the height of the partition with horizont members. The ends of the studs are usually secured betwee flat channel bars placed along the ceiling and the floor, a shown in Fig. 39. Heavy expanded metal is then laced t the upright channels. When an air space is desired in the partition, the expanded metal is placed on both sides of the channels, but more frequently the partition is made solid and the expanded metal is placed on one side of the studding only. The partition, when constructed solid, is about 2 inches thick, and as it is monolithic in character it offers great resistance, at the same time taking up little space. These partitions have been constructed more than 20 feet in height, and several hundred feet in length.

A recent modification of this construction does away with the vertical steel channels, the expanded metal being held temporarily by wooden studs, which are removed before the final coat of plaster is applied.

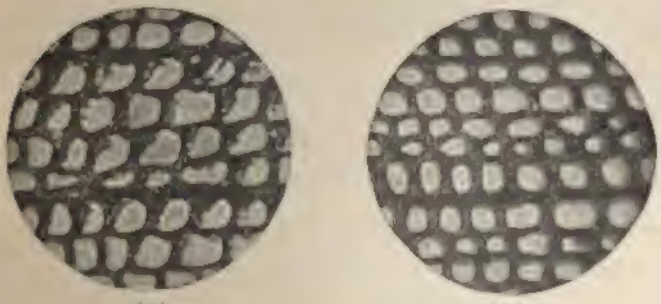
FIREPROOF WOOD

51. The necessity for a thoroughly **fireproof wood** as a fire-retardant and also as a means of reducing the amount of combustible material in a building, has been felt since the first efforts were made toward establishing a fireproof system of construction. There are many methods of making wood or timber fireproof; the most inefficient are those that attempt to accomplish the end by paints and superficial applications of fire-resisting chemicals, while the efficient methods are those that impregnate the fibers with some chemical that surrounds the wood cells and produces a coating that effectively protects the timber from combustion. This protection around the cellular wood tissues is shown in Fig. 40 (*a*), which is a photograph under the microscope of a section of fireproof wood magnified 225 diameters. By comparison with (*b*), which shows wood that has not been treated, the efficiency of the chemical coating is apparent. The superficial

treatment of wood is entirely inadequate for the purposes of reliable fireproofing, while its impregnation has been improved until it has been demonstrated that timber treated by the latest processes is worthy of consideration in important fireproof construction.

The principal difficulties encountered in the latter method were that the chemicals dried out and the wood became as inefficient as ever; they also, after a time, seriously impaired the strength of the wood and were inclined to make it difficult to work. Besides, the wood was made difficult to finish and was liable to shrink, warp, and twist.

52. The latest and apparently best process of fireproofing timber is known as the *electrical*. It consists of subjecting carefully selected timber to successive treatments of



(a) (b)

FIG. 40

saturation, evaporation, impregnation, and kiln drying. The timber is first rid of all sap and fermenting substance by saturation; it is then subjected to evaporation; afterwards it is rolled on a suitable truck into cylinders where it is carefully subjected to a solution of certain sulphates and phosphates of ammonia under a pressure of from 80 to 120 pounds per square inch, the pressure varying with the character of the timber and the closeness of its grain. Before being taken from the cylinder, the timber is carefully inspected and its weight before treatment compared with its weight after impregnation. In this way the percentage of

are withdrawn they weigh nearly twice as mu
went into the cylinders, and when they ar
usually contain about 6 ounces of the firepr
to the board foot. In the inspection, usu
planks are cut through the center to detern
solution has permeated the entire mass of tl

53. While tests demonstrated that th
described did effectually fireproof the timbe
first be determined whether the chemicals
wood was permeated had wearing properties
the timber so treated retained its fireproo
years of exposure and service. To determi
of the fireproof quality of wood treated
described, the United States government c
tests on samples of fireproofed white pine,
nut that were taken from the woodwork of
crew's and officers' quarters of the United
boat Winslow, after having been in place
tests were made before numerous and re
and are as follows:

"1. *Stove Test.*—A piece of 11-inch vitrifie
stood on end over bricks and a piece of i
secured at a height of about 10 inches below
were laid shavings and kindling wood saturat
Underneath, for the purpose of igniting
placed waste, also saturated with kerosene.
vings and kindling wood were placed two sp
pine, two of ash, and one of butternut, that h
from the Winslow, and had been subject
preparation than cutting to size suitable for

pecimens ceased to give off any flame or glow, and on emoval showed their surfaces to be charred to a depth varyng from $\frac{1}{8}$ to $\frac{3}{16}$ inch. These specimens, on being split p, showed their interiors to be entirely sound and free from ny evidences of charring, etc.

"2. *Hot-Plate Test.*—A steel plate $\frac{1}{2}$ inch thick and about inches wide by 2 feet long was heated to a bright redness n a furnace, and immediately after its removal a specimen f each variety of the fireproofed woods was laid upon its pper surface and allowed to remain in contact so long as there was any flaming from the wood. The ash specimen ceased o give off any flame first, the butternut specimen second, and the pine specimen last. On examination of the charred surfaces of these specimens that had been in contact with the plate, they were found to be charred on the surfaces only, and to be free from ash, and displayed, after removal, no tendency to glow.

"3. *Hot-Rivet Test.*—A number of rivets were heated in the furnace to a driving temperature and laid upon the clean surface of the fireproofed woods submitted. When first placed in contact some flame was given off from the wood, but this flame very soon disappeared, and on removal of the rivets there was no tendency to continue to glow; and the wood showed a limited charred effect, without any evidence of ash where the rivets had been in contact with it.

"4. *Splinter Test.*—This test consisted of holding splinters of the several woods received from the United States torpedo boat Winslow over the flame of a Bunsen burner, burning coal gas. In all cases, both when the slivers were held horizontally and when they were held vertically in the flame, a flame was given off from the specimens while in contact with the Bunsen flame, but immediately on removal each specimen ceased to flame or show any evidence of glowing, there being in no case any ash formed.

"The specimens above mentioned were subjected to no other preparation for this test than reduction to the sizes necessary and the removal from their surfaces of the paint or varnish with which those surfaces had been covered."

54. Besides the government tests of fireproof woods, the New York Building Department has made a thorough test as well, and the care with which it was conducted and the evident severity of the test is apparent in the following copy of the official record of the proceedings:

"A small frame structure 12 feet 4 inches by 18 feet 5 inches and 16 feet 4 inches high, divided into two rooms, had a cupola in the center 3 feet 2 inches by 6 feet, extending 10 feet 9 inches above the top of the main structure, which was also divided in the center, making a separate flue for each room. One of the rooms was left with the studding and rafters exposed, while the other had a mantel fireplace and paneled sides and ceiling, exhibiting the finish of the different kinds of wood used in trimming fireproof buildings. The structure was built at the foot of Nineteenth Street and East River, and on the day of the test was placed on a barge, the floor of which was protected by three layers of 1½-inch mahogany, and was towed to the foot of Eighty-sixth Street and East River, where the test was made.

"A small quantity of fuel was kept constantly ignited in the fireplace, and burned for several hours, with the effect of charring a small hole through the lower part of the mantel back.

"Fuel, consisting of shavings, pine cordwood, and barrel staves, well saturated with kerosene, was placed on top of the barge beneath the first tier of beams of the unfinished room, and being lighted and fire rekindled from time to time, burned for 35 minutes.

"Fuel, same as above, was also placed in the unfinished room next the partition, and being lighted and fire rekindled from time to time, burned fiercely for 21 minutes, after which time no flames were seen issuing from the top of the cupola, and the interior gradually cooled, leaving the wood charred. The building remained practically intact at end of test.

"From the results of the test as made, and from reading the reports on file in this Department, the general opinion is that wood treated by this process is a fire-preventive, and

may be used in buildings, in accordance with Section 105 of the Building Code."

55. Fireproof wood is a desirable material for the interior trim of office buildings, and as a fire-preventive should be more generally used. It has been demonstrated almost conclusively that the chemically treated wood is as desirable and can be finished as well as the natural wood. When the fireproof wood is well kiln-dried it is not liable to warp and shrink, and does not cause the corrosion of nails or screws, or hardware that may be attached to it. If any additional expense occurs, owing to difficulty in working fireproof wood over timber not chemically treated, it is not sufficient to make any appreciable cost. That fireproof wood retains its virtue after a lapse of years in service is conclusively proved by the government test quoted.

ROOF-TRUSS DESIGN

(PART 1)

CONDITIONS INFLUENCING DESIGN

1. Loads.—Roof trusses are especially adapted to such buildings as railroad terminals, opera houses, assembly halls, markets, etc., since they support the roof and still afford a clear space without the use of intermediate columns. They support loads of various kinds, such as the dead load, due to the weight of material used in constructing and covering the roof, the wind load, the snow load, and frequently the weight of plastered ceilings, as well as loads from attic floors and from suspended platforms and galleries.

The building laws of some cities require that trusses having a pitch, or slope, of less than 25° be designed to sustain a load of 50 pounds per square foot of actual roof area, and those of greater pitch to support a load of 30 pounds per square foot of area covered. In these values, ample allowance is made for snow and wind loads, in addition to the weight of material used for roof covering and construction. In laying out the stress diagrams, all these loads are considered, and both members and details are then proportioned to withstand the various stresses produced.

In supporting the roof covering, it is customary to introduce a secondary construction, which transfers the weight to the trusses. This usually consists of large beams, called *purlins*, that rest on and connect the trusses at their *panel points*, or points at which the main rafter and the web members intersect. These purlins support the rafters of the

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roof, on which the wood sheathing and roofing material are placed. In wood construction, a purlin usually consists of a single rectangular timber hung or placed on top of the trusses, or mortised or gained into the cheek of the rafter member, while in iron and steel construction I beams and Z bars are generally employed.

Great care should be taken to avoid, as far as possible, the accidental stresses to which trusses are so often subjected during erection, and which can scarcely be calculated.

2. Slope of Roof.—The character of the roofing material must be considered in determining the direction of the upper chord. For instance, when shingles are used, the pitch should not be less than 1 of rise to 2 of run, while with slate if the pitch is less than 1 to 3, the wind is liable to blow the rain under the slate, thereby causing leaks. A slope of 1 to 2, however, is preferable for slate, although, when occasion requires, the minimum pitch of 1 to 3 may be employed.

Corrugated iron is liable to leak if laid with a pitch of less than 1 to 3, while in a gravel and tar roof, if the slope is greater than 1 to 4, the heated tar is apt to run down and collect at the lower portion of the roof, leaving the upper part exposed and unprotected, and rain falling on such a roof flows off so quickly that the pebbles are washed out of the roofing. Flat clay tile set in asphalt may be used on flat roofs, but clay and metal tile simulating corrugated or Spanish tile are usually laid with a pitch somewhat greater than 1 to 2.

3. Distance Between Trusses.—The fact that the economy of the design is so largely dependent on the spacing of the trusses, makes it necessary that an effort be made to ascertain the distance that may most economically exist between them. This is especially the case when the building over which they are to be placed is of such a character that the spacing of the trusses governs, or, at least, affects the exterior design. Often, too, the engineer is restricted by the fact that the size of the lot must be considered when determining the size of the building, and hence, the distance

between the trusses is influenced to a certain degree, particularly when architectural effect is desired.

4. When the architectural design of the building is subordinate to the structural design, the distance between trusses may be most economical. This spacing is, however, somewhat difficult to determine, and no general rule can be given that holds true for specific cases. Extensive investigations, in which the material required and the cost of labor in both shop and field were considered, show that the most economical distance between trusses with spans of 40 to 200 feet is about one-fourth the span. Roof principals or trusses are seldom placed less than 10 feet and never more than 50 feet on centers. The usual span of the purlins for triangular trusses is from 12 to 20 feet, while arched trusses built in pairs and having purlin members in the form of latticed girders are seldom placed less than 25 feet or more than 50 feet apart.

5. **Material Used.**—The general design of a truss is influenced by the material employed in its construction, and the choice of material is influenced by its cost and availability, as well as by the span of the truss and the loads that come on it. When the span exceeds 80 feet, and the loads are comparatively heavy, steel is usually the best material to use; but if steel is unavailable, timber may be used for trusses having spans as great as 150 feet. When timber trusses have as great a span as this, they are usually arched, and built in pairs.

While steel may be used for the construction of trusses of all spans, great and small, the designer is frequently compelled to use timber because the cost of the work is limited. Timber trusses may be built entirely of wood, with the exception of the spikes and bolts used at the connections; they are adapted to localities far removed from industrial centers, where castings, special forgings, and steel shapes cannot readily be obtained. Ordinary timber trusses are built with wooden rafter and tie-members and struts, while the tension members extending from the foot of the struts are wrought iron or steel. When all the tension members

of a truss, including the tie-member, are steel or wrought iron, and the connections are made by pins, the type is known as a *composite truss*. In some composite trusses only the rafter members are timber, the struts being made of structural steel shapes latticed or tied together by clips.

6. After the general dimensions of the roof truss have been determined, if economy is to be considered, the cost must be investigated. Should the conditions admit a choice of several designs, it is often desirable to estimate the cost of each, and adopt the one whose construction costs the least.

In designing trusses it is generally cheaper to use stock sizes of timber or steel shapes, for by so doing, even though the members are of a larger size than actually required, the work is usually facilitated to such an extent that the time required in its performance is materially reduced, which is frequently a factor of the utmost importance. But if time need not be considered, and the work involves the use of large quantities of material, the design may usually be cheapened by using special sizes of timber or rolled shapes. The saving is due to the fact that the members of the frame may be proportioned with more exactness to withstand the stresses to which they are subjected, and thereby much less material is required. In most cases, however, the construction is not of sufficient magnitude to warrant the use of special sizes in either timber or rolled shapes, and the general practice is to give preference to stock sizes, using as small a variety as possible. Likewise, when the truss is being assembled and erected, labor is saved and complications are avoided in both shop and field by using, whenever practical, the same sized rivets, pins, and bolts throughout the frame.

TRUSS FORMS

7. King-Post Truss.—The simplest form of roof truss, as shown in Fig. 1 (*a*), is a triangular frame consisting of two equal rafter members connected by a tie-beam. The tie-beam becomes necessary when the outward thrust of the rafters is too great for the resistance of the walls. The

bending moment on the tension member $a\,b$, due to its own weight and the load of the ceiling, increases as the span of the truss increases. The section required to resist both the tensile and transverse bending stresses would necessarily be large; hence, to reduce the size of this member a suspension rod is introduced at $c\,d$, Fig. 1 (b), which makes the effective span of the tie-beam $a\,b$ equal to one-half the span of the truss. For this reason the span of the truss shown in (a) is limited to about 24 feet, while the construction shown in (b) may be used with spans as great as 35 feet. When the span

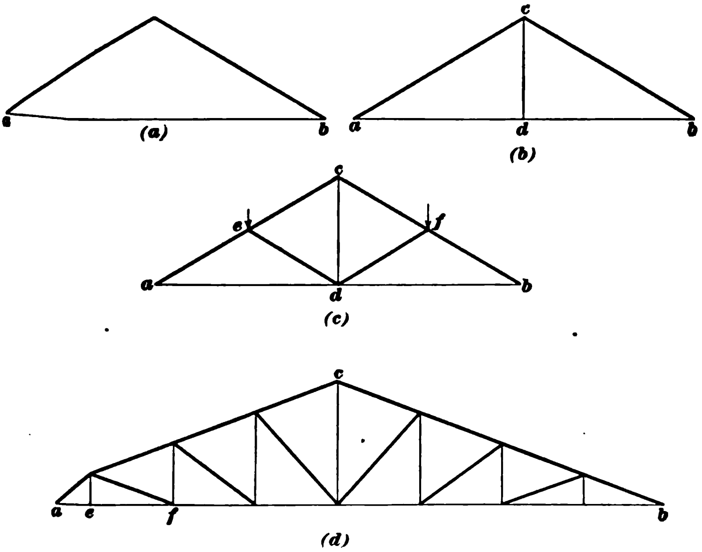


FIG. 1

is still further increased, it becomes desirable to introduce intermediate purlins at e, f, Fig. 1 (c). If these purlins were unsupported, a bending moment would be created on the rafter member at e and f; intermediate struts are therefore introduced at $e\,d$ and $f\,d$ to relieve the members of the transverse stress. Originally, the member $c\,d$, Fig. 1 (c), was made of wood and was known as a *king post*, or *king rod*, and the truss termed a **king-post truss**. But as there was some difficulty in making the connections, the form has

been modified by using a wrought-iron tension rod at *cd*. This form of truss may be adopted for frames having spans as great as 40 feet, but it is often found economical to employ the intermediate strut in triangular trusses of shorter span. Extra labor and material are necessitated by the introduction of the strut, but the reduction permissible in the size of the rafter member more than compensates for it.

8. Howe Truss.—The method of increasing the number of triangles, as shown in Fig. 1 (*a*), (*b*), and (*c*), may go on indefinitely and the natural outcome is the **Howe truss,** or **king- and queen-post truss,** as it is sometimes called. This truss, which is shown in Fig. 1 (*d*), may be extended to very large spans by increasing the number of panels, but it is not suitable for steep roofs, because as the span increases, the length of the struts toward the center becomes so great as to require timbers of large size. For long trusses the usual rise is one-seventh of the span. With long spans, it is customary to reduce the shear at the heel of the truss by placing the compression member nearest the wall more nearly vertical, as is shown at *a*, Fig. 1 (*d*). In this frame the vertical tie at *c* is not needed to resist any stress created in the frame, but is necessary to support the long extent of lower chord between *f* and *a*.

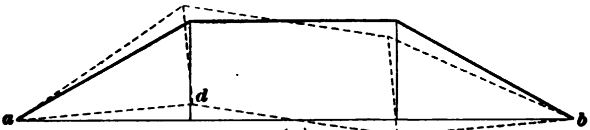
9. Queen-Post Truss.—The style of truss shown in Fig. 2 (*a*) is known as the **queen-post truss.** Its structural design is not as correct as the king-post, but it is very useful where a rectangular space is desired in the center of the room, as in an attic, or small hall.

When loaded unsymmetrically, it tends to assume the shape shown by the dotted lines; for this reason its use should be confined to cases where the loads are small and symmetrically placed. The whole tendency to resist an unsymmetrical load is directed in bending the tie-member *a b* at the points *d*, *d'*.

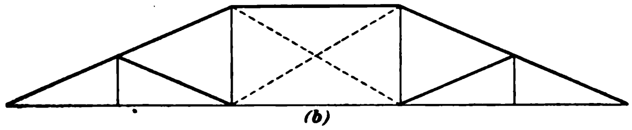
From Fig. 2 (*b*) it may be seen that the development of the queen-post truss is not as extensive as that of the king-post, although it may be used for larger spans when it is

cross-braced in the center, as shown by the dotted lines, but this defeats the primary use of the truss. When used without cross-bracing it should be strongly bracketed or braced with wrought-iron plates or knees at the angles of the rectangle.

10. Bow-String Truss.—Another form of truss commonly built of wood, and used for spans as large as 100 feet, is shown in Fig. 2 (*c*); this is known as the **bow-string truss.** Its upper chord may be either an arc of a circle or a parabola. The peculiarity of this design is that the intermediate cross-bracing under a vertical load sustains little or



(*a*)


(*b*)

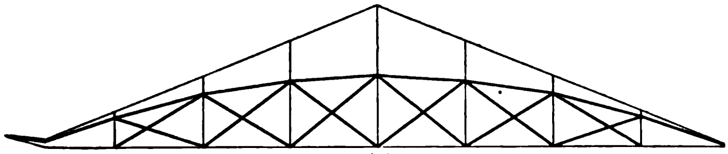

(*c*)

FIG. 2

no stress, according to the accuracy with which the curve of the upper chord coincides with the line of pressure of the loads. When these curves coincide, the cross-bracing serves to stiffen the design and to resist the strains due to eccentric and wind loads. When properly designed, the upper and lower chords take up all stresses from the vertical loads.

In order to utilize this form of truss, it is customary to extend the vertical members upwards to the required pitch line of the roof. By this means a roof of even pitch from apex to eaves may be supported.

[illegible] Truss.—The truss shown in Fig. 3 (*a*) is [illegible] Pratt; its similarity to the Howe truss, [illegible] be seen, and the points of difference [illegible]. In the Pratt truss, the compression members [illegible] the frame, or the struts *a*, *a* are vertical, while [illegible] *b*, *b* are oblique. In the Howe, a reversed [illegible] for the struts are oblique and the tension [illegible]. The Pratt offers rather a better appearance [illegible] Howe from the fact that the oblique members, [illegible] at different angles, are round bars that [illegible], but in the Howe they are made of [illegible] frequently unnecessarily heavy and far [illegible] appearance, while the vertical timber struts

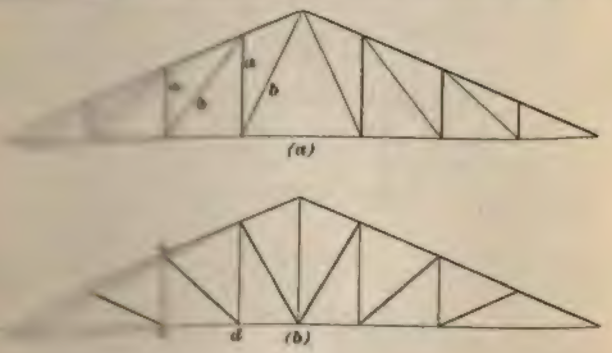


FIG. 3

[illegible] entirely unobjectionable. Also, in the [illegible] members, or struts, are shorter [illegible] is of advantage from the fact that [illegible] a given section, as *a*, Fig. 3 (*a*), need [illegible] compression only, a longer strut of the [illegible] Fig. 3 (*b*), is more apt to bend, and [illegible] sometimes be figured to withstand both [illegible] stresses. The Howe truss, however, [illegible] that the connections at *c*, *d*, and *e* are [illegible]

[illegible] Trusses.—The form of **church roof** [illegible] influenced by the architectural

eatment of the interior. The design usually includes a aulted or arched ceiling, and the lower chord of the truss ıust be so arranged as to permit this effect. Most church oofs have a pitch of from 45° to 60°, which gives sufficient eight for the arching or raising of the lower chord and lso produces less outward thrust on the walls. From the act that the walls of such edifices are usually buttressed and apable of resisting considerable thrust, the lower chord of he truss is relieved of much of the stress.

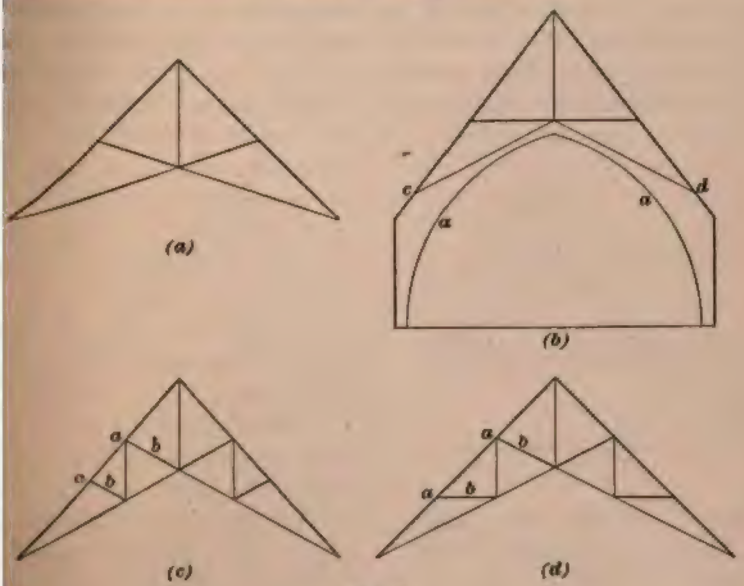


FIG. 4

The simplest type of church roof truss is known as the **scissors truss**; this, in its elementary form, is shown in Fig. 4 (*a*). It is a development of the king-post truss, with an intermediate strut, in which the lower member has been raised until the strut and the lower chord are in line. This truss, on account of its shape, is not suitable for large spans. A slight movement at the abutments or walls produces considerable distortion in the frame, greatly increasing the stress in the members.

Another form of the scissors truss is shown in Fig. 4 (*b*), in which the line of the vaulted ceiling is designated at *a a*. It is usual in planning ceilings of this character to form molded and projecting ribs beneath each truss, as such a treatment destroys the monotony of a plain ceiling and permits the lowering of the bottom chord or the tie-members of the truss.

Trusses built as shown in Fig. 4 (*b*) are apt to be weak at the connections *c* and *d*, because at these points there exists not only a compression in the extension of the rafter members, but a bending moment as well, due to the fact that the reactions about these points act with a lever arm equal to the perpendicular distance from their line of action to the point *c*. As this bending moment must be considered in designing the truss, and it is sometimes provided for with difficulty, it is best to avoid it, if possible, by connecting the tension members with the ends of the rafter members, as shown in Fig. 4 (*a*).

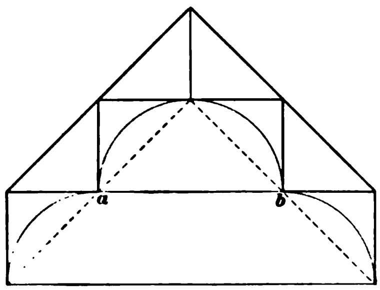


FIG. 5

With the increase of span, the scissors truss develops into the form shown in Fig. 4 (*c*) and (*d*). These types are satisfactory designs, and are applicable to spans as large as 60 feet. The intermediate purlins supporting the roof are located on the rafter member of the truss at the points *a*, *a*, the load being transmitted directly to the truss by means of the struts *b*, *b*.

In designing a truss of this character, it is always advisable to divide the upper chord into equal parts, because the appearance is more pleasing and the loads more uniform.

13. Hammer-Beam Truss.—A form of truss commonly used in church construction, shown in Fig. 5, is known as the **hammer-beam truss**. The lower chord is made up of curved pieces, which must be considered as

straight lines in solving the stress diagram. The form of the truss indicated by the dotted lines is assumed in the calculation, and the curved members are figured for both the tension and bending stress due to their being bent out of line.

Frequently trusses of this character are provided with a horizontal tie-member, extending from *a* to *b*. This member strengthens the frame considerably, and, when so constructed, these trusses may be used for spans up to 500 feet.

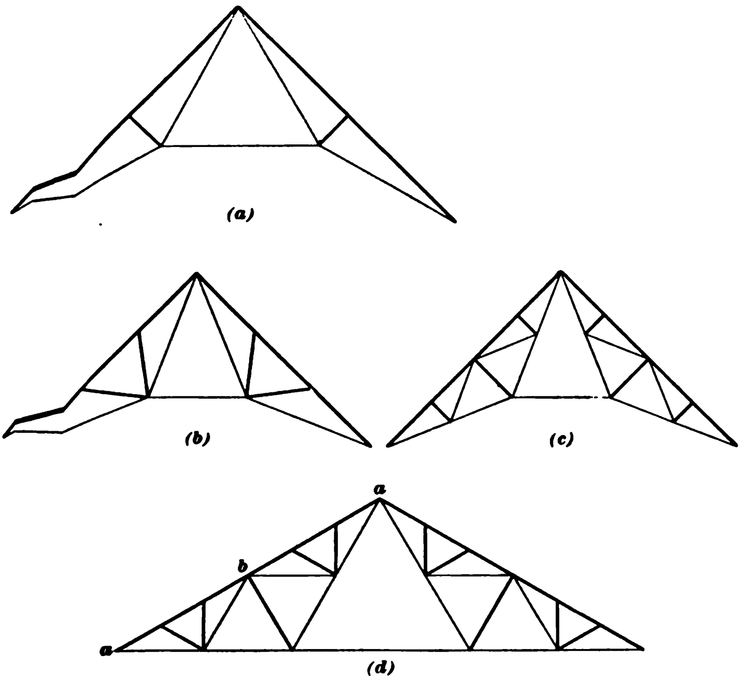


Fig. 6

14. Fink Truss.—The truss shown in Fig. 6 (*a*) is known as the **Fink truss,** from the engineer who first evolved the design. It is used frequently in roof construction, and on analysis may be considered as two trussed beams used as rafters, and connected at the lower ends of the struts by a tension rod or tie. The most common of the various types of Fink truss is provided with a straight tie or

lower chord, as shown in Fig. 6 (*d*). In order to gain headroom or height between the lower chord and the floor, the central member may be raised, as shown in (*a*), (*b*), and (*c*). The type in (*d*) may be used for spans as great as 100 or 125 feet. The main rafter member is supported between the points *a* and *b* by two secondary members instead of by one, as in the truss shown in (*c*).

The Fink truss may be of composite build. or may consist entirely of steel, but it is not adapted to construction entirely of wood. When made wholly of steel it is more economical for long spans than either the Howe or the Pratt, especially where the roof loads are light and there is no ceiling load, as in shops, car barns, etc.

15. Various Forms.—In addition to the forms of roof trusses described, many types are in use; a few of these are illustrated in Fig. 7. The form in (*a*) is a Howe truss that makes a frame of good appearance and provides increased headroom by having the lower chord raised at the center.

Gambrel roofs with spans varying from 40 to 60 feet are sometimes supported by trusses of the form shown in Fig. 7 (*b*). The suspension support for a flat ceiling is shown by the dotted lines. Trusses whose upper or compression chord is made up of several members extending in different directions from the various panel points, are of interest from the fact that the upper chord frequently approaches the form that would be assumed by the funicular or equilibrium polygon, and, consequently, where the lower chord is horizontal, the stresses in the web members *a*, *a* are eliminated when the truss is symmetrically loaded.

In Fig. 7 (*c*) is shown a quadrilateral truss built on the lines of a Howe, and supporting on its upper chord light scantling work for the sloping roof. A truss of this design can be used for large spans, and is particularly adapted for roofs over large halls, shops, shipways, and docks. Counterbraces should be provided, at least in the panels *a*, *a*, to prevent any distortion liable to be produced in the frame by unequally distributed snow loads and wind loads, although

they are not absolutely required if all the members in the truss are capable of resisting both tension and compression.

Roof covering for seats in the open, as at ball parks, tracks, and other outdoor places of amusement, may be constructed as shown in Fig. 7 (*d*). In this case the overhanging portion of the frame supports the roof over the front portion of the staging, where the choice seats are located. The overhanging, or cantilever, roof framing provides a support

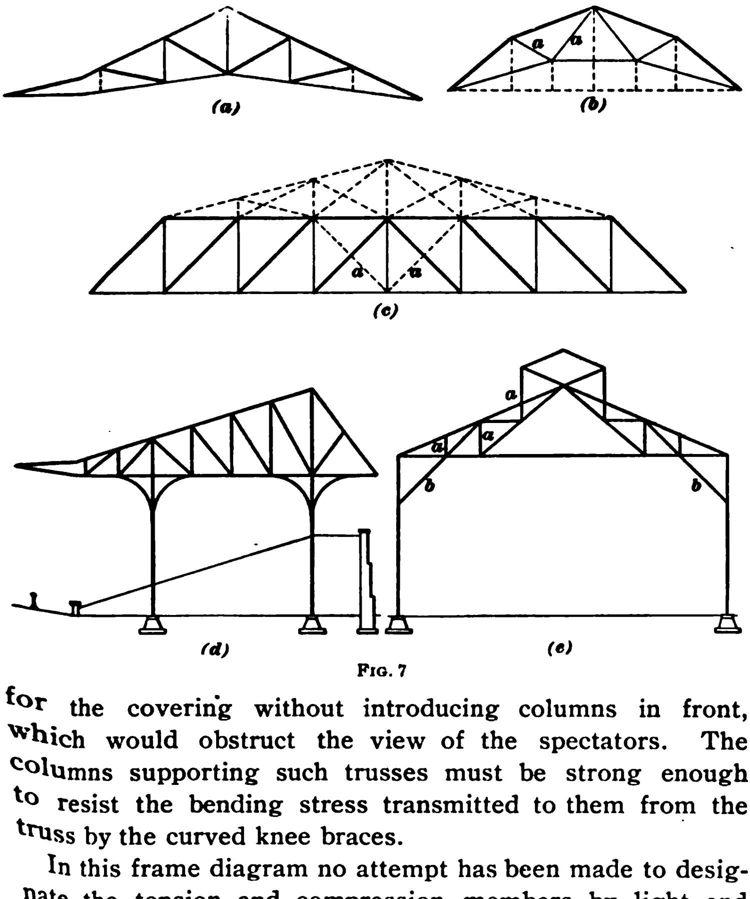


FIG. 7

for the covering without introducing columns in front, which would obstruct the view of the spectators. The columns supporting such trusses must be strong enough to resist the bending stress transmitted to them from the truss by the curved knee braces.

In this frame diagram no attempt has been made to designate the tension and compression members by light and heavy lines, since the character of the stress changes with the

chords of that portion of the frame betw
and all these stresses are subject to chang
considered as blowing under the roof with

The modified Fink truss shown in Fig.
for mill buildings and shops. The struts
vertically instead of normal to the slope, a
provided at b, b to render the frame more
a wind brace, one being in tension and
pression when the wind acts from either s

It is not customary to provide the colu
truss with heavy foundation piers and fo
sider them as fixed at the ends, which is
in determining the stresses in the fran
must be proportioned to withstand a co
moment, which is created by the lateral
them by the knee braces.

16. Comparison of Forms.—In F
and (*d*) are shown four styles of trusses
of the same pitch and sustain loads of t
being provided with a tie-rod. These illu
Howe, Pratt, Fink, and bow-string tru
Considering the first two, the maximum
member and also in the horizontal tie-r
equal, respectively, to those in (*b*), but a
exists in the amount of stress in the sever
the two trusses, the Howe displaying a
over the Pratt. The latter, however, is su
in that the diagonal *L M* is subjected to a te

STRESS AND MATERIALS

STRESS

17. Strength of Parts.—The members of a truss may be subjected to tension, compression, bending, or shear, and in some cases to more than one of these stresses. When possible to avoid it, members that are subjected to tension and compression alternately should not be called on to resist cross-bending also, since in such a case one of the primary objects of truss construction, that is, the reduction of cross-bending to direct tension and compression, will be defeated.

18. In proportioning a member for tensile stress, the following rule is applicable:

Rule.—*Divide the tension to be resisted by the allowable stress per square inch; the quotient will be the required net section of the member. To this add an area sufficient to allow for any cutting or notching of members that may be necessary.*

The amount to be added is given under the separate heads in designing. In the case of tension members, it is well to support the long pieces at several places in order to relieve them of the extra strain they must withstand in supporting their own weight. This does not apply so much to slanting or vertical ties as it does to the horizontal members.

In proportioning compression members made of wood, it is customary to assume one dimension of cross-section, and if made of structural steel shapes, the radius of gyration is assumed. The total compression in the member divided by the allowable unit stress gives the required area for the member. There are many values for the allowable compression per square inch of timber, but the ones adopted for this Section are given in *Materials of Structural Engineering*, Part 3.

19. Compression and Tension Members Under Cross-Bending.—All members, except vertical ties and struts, are to a certain extent subjected to cross-bending as

well as to direct stress. These bending moments occur particularly: (1) When members placed in a horizontal or inclined position are required to support their own weight; (2) when the neutral axis of the member does not pass through the center of gravity of the connection; (3) when the member must support loads between its panel points.

20. Obtaining Bending Moment of Truss.—Sometimes the cross-bending stress due to the weight of the member may be counterbalanced by placing the centers of the pin or end connections out of line with the neutral axis of the member. If this is not done the amount of stress due to the two causes should be calculated and an area that is sufficient to reduce this stress to the safe limit should be added. There are three methods of calculating the bending moment.

First Method.—The simplest method consists in assuming a depth for the member, then calculating a width sufficient to withstand the bending moment, and adding to the area obtained by multiplying the width by the depth, an amount sufficient to provide for the direct stress. This is illustrated in the following example:

EXAMPLE.—What size timber will be required for a vertical compression member 11 feet $2\frac{1}{4}$ inches long that is subjected to a uniform load of 10,800 pounds, and a direct compression of 90,000 pounds?

SOLUTION.—The bending moment on a beam uniformly loaded and supported on both ends may be determined by the formula $M = \frac{WL}{8}$; hence, in this case, $M = \frac{10{,}800 \times 11.1875}{8} = 15{,}103\frac{1}{8}$ ft.-lb.; or, multiplying by 12, 181,237.5 in.-lb. The modulus of rupture of yellow pine is 7,300 lb.; then, assuming a factor of safety of 4, the safe working value of this material is $7{,}300 \div 4 = 1{,}825$ lb. per sq. in. The required section modulus for the rafter member is found by dividing the bending moment, 181,237.5 in.-lb. by 1,825. The quotient obtained is 99.3. The section modulus of any rectangular beam is equal to $\frac{bd^2}{6}$, so that if the depth of the member is assumed to be 14 in. the width or breadth required can be obtained by substituting in the formula $\frac{bd^2}{6}$, or $99.03 = \frac{b(14)^2}{6}$, from which b is found to equal 3.03 in.

The direct compression on the rafter must be considered next. If the breadth of the member is assumed to be no less than one-fifteenth

the length, a value of 1,000 lb. per sq. in. may be adopted. 90,000 ÷ 1,000 = 90 sq. in., the area required to resist this stress. As a depth of 14 in. has already been considered, the additional width required in the rafter will be 90 ÷ 14, or, approximately, 6.4 in., which, added to the width already calculated, gives the theoretical size of the member as 6.4 + 3.03 = 9.43 in. As there is no commercial size manufactured that is 9.43 in. by 14 in. in section, a 10 in. by 14 in. may be used with advantage, since the extra section compensates for the part cut away. Ans.

Second Method.—In this case the size of the member required to resist the direct stress is found. The maximum fiber stress per square inch due to the bending moment is then calculated for a member of this size, and added to the direct stress per square inch. The dimensions of the member need not be changed unless the sum of these stresses is more than 25 per cent. greater than the maximum allowable fiber stress under a direct load, in which case a larger size member should be adopted and the stresses must be again calculated.

EXAMPLE.—What size timber will be required to resist a compression of 33,400 pounds and a center load of 500 pounds, providing its length is 12 feet?

SOLUTION.—To approximate the size required, 1,000 lb. per sq. in. is assumed as the allowable stress. Then the area required would be $\frac{33400}{1000}$, or 33.4, and a section 6 in. by 6 in. would be used; but if this were the case, the length would be twenty-four times the width, and hence the calculation must be done by using the formula. Assuming the least dimension of the timber as 6 in., the allowable compression per square inch is $s_a = 825 - .175\left(\frac{l}{d}\right)^2 = 825 - .175\left(\frac{144}{6}\right)^2 = 825 - 100.8 = 724$ lb.

Dividing the compression, 33,400, by this value, the required area of the timber is 33,400 ÷ 724 = 46.1, which can be provided for by a 6″ × 8″ timber. The extreme fiber stress due to bending is $\dfrac{\dfrac{500 \times 144}{4}}{\dfrac{6 \times 8^2}{6}}$ = 281 lb. Since the greatest allowable compression under direct loads is 724 lb. per sq. in., the greatest allowable stress under the combined stress is $724 \times 1\frac{1}{4}$ = 905 lb. per sq. in. The stress due to end compression is 33,400 ÷ 48 = 696 lb. The sum of 696 and 281, the maximum fiber stress due to bending moment, equals 977 lb., and since this exceeds the allowable limit, the size must be increased and the stress recalculated.

Third Method.—In the two methods given, the bending moment due to the direct load multiplied by the deflection of the member under a bending moment is neglected. In some cases this is very great, as in a compression member the external bending moments tend to force the member out of line, giving it an initial deflection. This deflection forms an arm into which the direct compression acts, tending to bend the member still more, and thus increase the maximum fiber stress. It is of greater importance that this be considered in compressive members than in tension members, since in the latter case it is counteracted by the tension.

The method of calculating by the *third method* is given by the following formula, in which

M_1 = bending moment at point of maximum deflection, from cross-bending external forces and from eccentricity of position of longitudinal loading;

d = maximum deflection of member from all causes acting simultaneously;

W_t = total direct loading on member, tension or compression;

M_2 = bending moment from direct loading W_t into its arm $d = W_t d$;

M = total bending moment $= M_1 + M_2$;

A = area of sections;

s_1 = unit stress on extreme fiber from bending alone at section of maximum bending moment; or of maximum deflection, as the case may be, in pounds per square inch;

l = length of member, in inches;

c = distance from neutral axis to extreme fiber on which stress from bending is s_1;

E = modulus of elasticity;

I = moment of inertia of cross-section;

s_2 = unit stress in member from direct loading, supposed to be uniformly distributed $= \frac{W_t}{A}$;

s = total maximum unit stress in extreme fiber $= s_1 + s_2$.

The total bending moment is then equal to $M = M_1 \pm M_2$, in which the positive sign refers to members under compression, while the negative sign refers to members under tension. The value of s_1 may be calculated by the formula

$$s_1 = \frac{M_1 c}{I \pm \frac{W_t l^2}{10 E}} \qquad (1)$$

in which the positive sign is used for compression members and the negative sign for tension members.

All dimensions given above should be taken in inches and all the forces in pounds. The maximum fiber stress s on the member is equal to the stress s_1 plus the stress due to the direct load s_2. This is illustrated by the following examples:

EXAMPLE 1.—What will be the maximum fiber stress due to bending and direct load on a horizontal tension bar 6 inches by $1\frac{1}{2}$ inches, whose length between pin points is 20 feet, and on which there is a direct tension of 126,000 pounds?

SOLUTION.—The weight of the bar is taken as .28 lb. per cu. in.; entire weight is $6 \times 1\frac{1}{2} \times 240 \times .28 = 604.8$ lb. The bending moment, therefore, is equal to $\frac{W l}{8} = \frac{604.8 \times 20 \times 12}{8} = 18{,}144$ in.-lb. Since the beam is rectangular in section, c, or the distance from the neutral axis to the extreme edge, is equal to $\frac{h}{2} = \frac{6}{2}$, or 3 in. I, or the moment of inertia of a rectangular beam, is equal to $\frac{b h^3}{12}$, which in this case is $\frac{1\frac{1}{2} \times (6)^3}{12}$, or 27. E equals 28,000,000; W_t equals 126,000. Then,

$$s_1 = \frac{18{,}144 \times 3}{27 + \frac{126{,}000 \times (240)^2}{10 \times 28{,}000{,}000}} = \frac{54{,}432}{52.92} = 1{,}028 \text{ lb., approximately}$$

The direct stress due to tension $= s_2 = \frac{W_t}{A} = \frac{126{,}000}{6 \times 1\frac{1}{2}} = 14{,}000$. Hence, the maximum fiber stress is equal to the sum of these two loads, or $14{,}000 + 1{,}028 = 15{,}028$ lb. Ans.

EXAMPLE 2.—In the above example, what would be the maximum fiber stress if the first method were employed; that is, adding together the stress due to bending moment and that due to tension?

SOLUTION.—The stress due to bending moment equals 18,144, while that due to tension equals 126,000. $126{,}000 + 18{,}144 = 144{,}144$ lb.

$$s_2 = \frac{W_t}{A} = \frac{144{,}144}{1\frac{1}{2} \times 6} = 16{,}016 \text{ lb. Ans.}$$

EXAMPLE 3.—What is the difference in tensile strength between a 5″ × 1″ tension bar 20 feet long placed flat and the same bar placed edgewise, provided that the maximum stress is not to exceed 15,000 pounds per square inch?

SOLUTION.—The weight of the member is $5 \times 1 \times 240 \times .28 = 336$ lb. The bending moment is $\frac{336 \times 240}{8} = 10{,}080$ in.-lb.

$$s_1 = \frac{10{,}080\,c}{I + \frac{W_t\,l^2}{10\,E}};\quad \frac{W_t\,l^2}{10\,E} = \frac{75{,}000 \times 240 \times 240}{10 \times 28{,}000{,}000} = 15.4$$

In the first case, $c = \frac{1}{2}$ and $I = \frac{5 \times (1)^3}{12} = .4167$, making

$$s_1 = \frac{10{,}080 \times \frac{1}{2}}{.4167 + 15.4} = 319 \text{ lb.}$$

Then the tensile strength of the bar placed flat is $15{,}000 - 319 = 14{,}681$. $14{,}681 \times 5 = 73{,}405$ lb., the strength of the bar.

In the second case, $c = 2\frac{1}{2}$ and $I = \frac{1 \times 5^3}{12} = \frac{125}{12}$, or 10.42, making $s_1 = \frac{10{,}080 \times 2\frac{1}{2}}{10.42 + 15.4} = 976$ lb. Hence, the tensile strength of the bar per square inch, when placed flat, is $15{,}000 - 976 = 14{,}024$ lb., and $5 \times 14{,}024 = 70{,}120$ lb.

The difference, in favor of the bar placed flat, is therefore $73{,}405 - 70{,}120 = 3{,}285$ lb. Ans.

EXAMPLE 4.—What will be the maximum fiber stress on a horizontal pin-connected compression member composed of two 10-inch 15-pound channels 18 feet long, placed back to back, supporting a compressive load of 100,000 pounds?

SOLUTION.—In the formula $s_1 = \frac{M_s\,c}{I - \frac{W_t\,l^2}{10\,E}}$, $W = 15 \times 18 \times 2 = 540$ lb.; $M_s = \frac{W\,l}{8} = \frac{540 \times 216}{8} = 14{,}580$ in.-lb.; I, as given in the handbooks, for one channel, is 66.82; therefore, for two channels it is 133.64. $\frac{W_t\,l^2}{10\,E} = \frac{100{,}000 \times 216 \times 216}{10 \times 28{,}000{,}000} = 16.66$; $c = \frac{h}{2} = \frac{10}{2} = 5$; $s_1 = \frac{14{,}580 \times 5}{133.64 - 16.66}$ $= \frac{72{,}900}{116.98} = 623.183$; $s_2 = \frac{W_t}{A} = \frac{100{,}000}{8.8} = 11{,}363.636$;

$$s = s_1 + s_2 = 623.183 + 11{,}363.636 = 11{,}986.919 \text{ lb. Ans.}$$

EXAMPLE 5.—What will be the difference between the maximum fiber stress calculated by the first method, and that calculated by the third method for the above compression member with a concentrated load of 3,500 pounds at the center?

Solution.—In this case the total bending moment on the compression member is equal to the bending moment due to the weight of the member plus that due to the external load of 3,500 pounds at the center, or $\frac{Wl}{8} + \frac{3,500\,l}{4} = 14,580 + \frac{3,500 \times 216}{4} = 14,580 + 189,000$ $= 203,580$ in.-lb.; $c = 5$ in., and $I - \frac{Pl^2}{10E} = 117$, approximately. Thus, $s_1 = \frac{203,580 \times 5}{117} = \frac{1,017,900}{117} = 8,700$; $s_2 = 11,363.636$, as before; $s_1 + s_2 = 8,700 + 11,363.64 = 20,063.64$ lb. maximum fiber stress.

The maximum fiber stress due to the bending moment, when considered as acting alone, is equal to the total bending moment as calculated, divided by the section modulus of the member. The section modulus for a 10-in. 15-lb. channel is 13.4, then for two channels it is 26.8. $203,580 \div 26.8 = 7,596.26$ lb. This, added to the direct compression, equals $7,596.26 + 11,363.64 = 18,959.9 = 18,960$. The difference in results of the two methods of calculation is $20,063.64 - 19,560 = 1,103.64$. Ans.

In many cases it is very important that the stress due to the bending moment be considered in connection with the compressive stress in the member, especially where the compression members are designed to carry concentrated loads.

MATERIALS OF CONSTRUCTION

21. Timber.—The materials usually employed in the construction of roof trusses are wood, cast iron, wrought iron, and steel. Steel is probably used most, but is sometimes so expensive that for the sake of economy, especially in non-fireproof buildings, it is well to substitute **timber.** Since such timber must be tough, strong, and durable the varieties available are necessarily limited. Georgia long-leaf yellow pine is the best that can be obtained, and is especially adapted to the construction of trusses on account of its uniform grade, as well as its high tensile and compressive strength. The Douglas, Oregon, and Washington fir, or pine, are also very strong and hence may be very advantageously employed in roof construction, while spruce, hemlock, and northern, or short-leaf, yellow pine are used to some extent, although hemlock, having a low tensile and compressive strength, is useful only in trusses of short span that are called on to

sustain but little stress. White pine is seldom specified for truss construction, as it is getting expensive and does not readily resist compression. White oak is excellent for keys bearing blocks, pins, etc., because it is tough and strong, but care must be taken concerning its location, since it shrinks considerably in seasoning. It can readily be seen that all woods last longer when protected from the weather, and for this reason a smaller factor of safety may be assumed for wood employed in heated buildings and other positions where it is not readily affected by dampness and changes of temperature.

TABLE I

COMMERCIAL SIZES OF TIMBER

Inches	Inches	Inches	Inches
1 × 8	2 × 10	4 × 6	8 × 8
1 × 12	2 × 12	4 × 8	8 × 10
1 × 16	3 × 4	4 × 10	8 × 12
1 × 18	3 × 6	4 × 12	10 × 10
2 × 3	3 × 8	6 × 6	10 × 12
2 × 4	3 × 10	6 × 8	10 × 14
2 × 6	3 × 12	6 × 10	12 × 12
2 × 8	4 × 4	6 × 12	

NOTE.—The size 10 × 14 may be obtained, but is not usually kept in stock.

22. Timber is made in certain stock sizes, as indicated in Table I, and although it is possible to obtain odd sizes, they are usually more expensive, as they are made by cutting down the stock next larger in section. Then, too, some stock sizes are cheaper than others, so that in many instances a considerable saving may be effected by arranging the design so as to use these cheaper timbers.

23. Influence of Shrinkage and Warping in Timber.—Since the rigidity of joints and connections is largely dependent on the warping, shrinkage, and twisting of the

material employed in truss construction, these matters must be given the earnest and careful study of the designer.

No woods are exempt from swelling caused by atmospheric changes, nor from shrinkage, which occurs during the process of seasoning; the shrinkage usually amounts to $\frac{1}{4}$ or $\frac{3}{8}$ inch per foot of width for ordinary commercial timber employed in roof trusses and subjected to the drying influence of a heated interior. The shrinkage lengthwise of the material is but slight, and is exceeded to a considerable extent by that which takes place in the width and thickness. The woods most affected are hickory and oak—especially the red oak, those least affected being soft pine, spruce, cedar, and cypress, while hard pine shrinks more than soft pine. When bolts are used as a means of fastening, care must be observed that the thickness of cross timber through which they extend is reduced to a minimum, and that no fastening camber rod or steel or iron tie is used unless some means of adjustment, such as threaded ends with nuts and washers or turnbuckles, be provided to take up any shrinkage that may occur after the frame has been in place and subjected to the atmosphere of the building for some months.

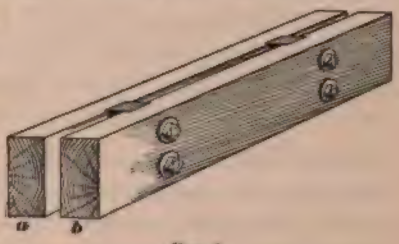


FIG. 9

All large-size timbers, particularly yellow pine, when not properly and completely seasoned, are liable to warp and twist, for which reason it is never advisable to use such material for important members, as the rigidity of the connection will be sacrificed and the strength decreased considerably. When there is a possibility of the material warping, important members may, with advantage, be built up of timbers 3 to 5 inches thick and kept about 2 inches apart by means of separators, as shown in Fig. 9. Since in such construction, members of comparative thinness are employed, it is possible to secure material whose quality is known, and the

formerly made of cast iron, but owin
this material is not very reliable when s
of stress and of temperature, its use h
bearing plates, heel connections, and s

25. Wrought Iron.—Wrought iro
form of commercial iron, is more readil
and acids than either cast iron or ste
a very tough, strong material of unif
used in truss construction for bolts, ri
and often for tie-rods, and is particula
as tension members. At one time stru
made of this material, but since steel,
by the modern process, is much cheape
wrought iron, these rolled sections are
made of steel.

26. Steel.—Steel is now employed
construction of all kinds, and being so
wrought iron, the various pieces may be
lighter. As in the case of timber, certa
are always kept in stock, by the use of
may be executed with the expenditure
money than when special sizes are ordere
ufactured by the various steel companie
handbooks that they publish. Steel is ma
of shapes, such as angles, I beams, Z bar
deck beams, flats, and rounds. Angles a
used as truss members, since they are ch
easily rolled, and can be handled with g
than the other shapes. I beams and Z b
purlins, while in the larger trusses stru

serve as tension members, especially in composite and pin-connected trusses. The pound price of steel is practically uniform for the usual rolled shapes, but channels and T bars are ordinarily more expensive than the others.

FACTOR OF SAFETY

27. Since due allowance must be made for unforeseen and unknown defects in materials and workmanship, and for unknown stresses that are liable to occur, the several parts of a structure must be proportioned to safely resist forces much greater than those obtained from the stress diagram. However, the stresses in roof trusses can be calculated with greater certainty than those in bridges or machines, as their application is more steady in its nature, and therefore not so severe on the material. Hence, in the design of roof trusses, unit stresses are permissible that are 50 per cent. in excess of those considered allowable in first-class bridges.

28. To provide greater ease in calculation, the materials and their values that are to be used are given in the tables for strength of materials. The allowable stress per square inch of material can readily be obtained by dividing the ultimate strength per square inch, as given in the table, by the factor of safety to be employed.

The strength of tension members can be conveniently obtained, but that of long columns must be calculated by substituting in the formula, while the values for short columns may be taken from the tables. In allowing for compression per square inch perpendicular to the grain, it is customary to use such strength values that the indentation of the wood will not exceed $\frac{1}{100}$ inch. This has been taken into consideration in determining the compressive strength of yellow pine perpendicular to the grain, as given in the various tables.

TRUSS DESIGN

29. After it has been decided what form of truss is to be used, and the intensity of the stresses has been obtained from the stress diagrams, it is necessary to calculate the sizes of the compression and tension members, and to design the connection. If the truss is of timber the size of the main rafters is especially important, since they are the principal compression members, and on their section the entire design of the truss is largely dependent.

In detailing joints and connections, the rule that axial lines of all members shall intersect at a single center at each joint or connection should be observed in all possible instances, as deviation therefrom to any considerable extent will cause bending moments in some of the members large enough to necessitate an increase in their size. When the parts used are rectangular in form, whether composed of steel or of timber, the axial line coincides with the center line of the member, and, of course, passes through the center of gravity of each cross-section, but when rolled sections are used, as, for instance, two angle irons riveted together, the center of gravity of the section must be located and the axial line made to pass through it.

TIMBER-TRUSS DESIGN

MEMBERS

30. Compression Members.—In order to obtain the section required for a **compression member** its length and the stress to be resisted must be known. The length, which is the distance between panel points, can be obtained from the frame diagram of the truss by scaling.

In most wooden trusses the upper chord is made in one piece from heel to peak, the size being determined by the

greatest stress. There is no economy in making the upper chord of separate pieces proportioned to the stress in each panel, because the extra bolts and the increased amount of labor required in making the joints cost more than the additional material made necessary by having the upper chord in one piece. Then, too, it is impossible to make a joint in a timber that will be as rigid as a continuous piece. The quantity of material saved by exactly proportioning the upper chord for the stress between panel points is illustrated in Fig. 8 (*c*), where the difference between the compression in the first and in the last panel of the Fink truss shown is 7,500 pounds, which represents a resistance of only 7.5 square inches of timber section, if an allowable unit compressive value parallel with the grain of 1,000 pounds is assumed.

31. It is difficult to determine the section required for a compression member of a given length to withstand a certain stress. The following method of approximating its dimensions is recommended: Assume a value for the ratio between the length of the column and the least dimension of the section of the column, both in inches, or for $\frac{l}{d}$, which will be the usual value for timber columns or posts. In most columns or struts this value $\frac{l}{d}$ is not less than 10, and no timber should be used as a compression member whose length is more than forty-five times its least dimension. The formula most frequently used for determining the strength of timber columns or struts is as follows:

$$u = s_c - \left(\frac{s_c\, l}{100\, d}\right) \qquad (2)$$

in which u = ultimate compressive strength of strut per square inch of section;

s_c = ultimate compressive strength of material per square inch parallel to grain;

l = length of column, in inches;

d = dimension of least side of column or post, in inches.

When the size of the strut section is to be determined, a given value, say 18, may be used for $\frac{l}{d}$, and the value of u found by substituting in the formula as follows:

$$u = s_c - \left(\frac{s_c}{100} \times 18\right)$$

or $$u = s_c - (.18 s_c) = .82 s_c \qquad (3)$$

For example, consider that the timber composing a strut supporting a load of 80,000 pounds has a safe unit resistance of 1,000 pounds per square inch, and that it is necessary to decide on the approximate sectional area of the strut before even the details can be designed, or the actual working resistance of the strut found. Substituting in formula (3), $u = .82 s_c$, or $.82 \times 1{,}000 = 820$ pounds. The total stress in the member, 80,000 pounds, divided by 820 pounds, the safe unit stress, gives a required sectional area of 97.56 square inches. Considering this column as square in section, the length of the side will be equal to the square root of this area, or 9.88 inches, which may be considered as 10 inches.

If a timber of square section is not desired, one side may be assumed and the other determined by dividing the approximate area required by the length of the assumed side. The general practice is to use timbers whose sectional dimensions do not differ by more than 1 or 2 inches for the smaller sizes, and by 3 or 4 inches for the larger. In case the dimensions calculated do not correspond with any commercial size of timber, the size next larger should be adopted. Even when the calculations require an exact size of timber, it is customary to adopt the one next larger to allow for the shrinkage, dressing, and cutting that the timber must sustain during its manufacture and erection.

32. In order that the application of the above formulas and calculations may be clearly understood, the following example is given:

EXAMPLE.—A strut 16 feet long is required to resist a compressive stress of 20,000 pounds. If the strut is built of yellow pine, having an

allowable unit compressive value parallel with the grain of 1,000 pounds, what size of square timber will be required?

SOLUTION.—The approximate size of the timber required for the strut may be determined by applying formula **3**, $u = .82\ s_c = 820$ lb. This result is the approximate safe unit stress, in pounds, of the column section, so that the area of column section required is $20{,}000 \div 820 = 24.39$ sq. in. As the strut is to be square in section, the approximate dimension of the side of the strut will be $\sqrt{24.39} = 4.94$ in., or nearly 5 in. The commercial size of yellow pine of square section nearest to this dimension is 6 in. by 6 in., which when dressed will be reduced to about $5\frac{3}{4}$ in. by $5\frac{3}{4}$ in. Considering the strut as a wooden column $5\frac{3}{4}$ in. square and 16 ft., or 192 in., long, the approximate size having been calculated, it is now necessary to determine whether the strut will safely sustain the imposed load of 20,000 lb. The desired result is obtained by applying formula **2**, in which $u = s_c - \left(\frac{s_c\, l}{100\, d}\right)$; by substituting,

$$u = 1{,}000 - \left(\frac{1{,}000 \times 192}{100 \times 5.75}\right) = 667 \text{ lb.}$$

The sectional area of a strut 5.75 in. square is 33.06 sq. in., and if 667 lb. is the allowable unit compressive value of the strut, its entire safe resistance is $33.06 \times 667 = 22{,}051$ lb. From this result it is evident that a strut made of $6'' \times 6''$ timber dressed on four sides will be strong enough. Ans.

33. Although it is not customary to splice the upper chord of a timber truss, it is sometimes done in Howe trusses where the difference in stress throughout the panel section is great. The construction commonly adopted in this case is shown in Fig. 10, where a timber of uniform size, large enough to resist the stress in two of the upper panel sections of the truss, is used from peak to heel. The increased stress in the two lower panels is resisted by a separate piece of timber, which is securely bolted to the rafter member and into which the struts *a*, *a* are butted or framed. In explanation of this: The size of the member required for the upper chord is determined by the stress in the member *DK*, the excess of the stress in *BG* being resisted by the reenforcing piece *bb*.

EXAMPLE.—What size timbers will be required for the rafter member in the Howe truss shown in Fig. 8 (*a*) if the main rafter member is reenforced through the two lower panels?

SOLUTION.—The short strut, or reenforcing piece, is required to extend through the first two panel points. The greatest compression on the two upper panels of the main rafter member is 25,000 lb., so that this stress determines the size of the member. The truss is assumed to be built of yellow pine having an allowable unit compressive stress of 1,000 lb., and the approximate size of the timber may be determined by formula **3**. By the application of this formula, it is found that the approximate sectional area required is equal to 30.48 sq. in. To attain this sectional area it is necessary to use a timber that, when dressed, will not be less than $5\frac{1}{2}$ in. by $5\frac{1}{2}$ in. Table I shows that the nearest commercial size to $5\frac{1}{2}$ in. by $5\frac{1}{2}$ in. is 6 in. by 6 in., which will be reduced, by dressing, to $5\frac{3}{4}$ in. by $5\frac{3}{4}$ in. The

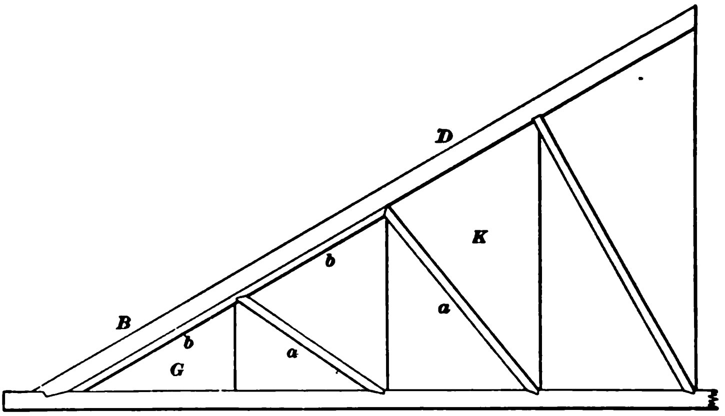


FIG. 10

unsupported length of the rafter member between panel points is 9.22 ft., or 110.64 in., so that by applying formula **2**, the value $u = 1{,}000 - \left(\frac{1{,}000}{100} \times \frac{110.64}{5.75}\right) = 808$ lb. The area of the rafter member being 5.75 in. by 5.75 in. = 33.06 sq. in., the safe resistance of the piece is $33.06 \times 808 = 26{,}712$ lb. The greatest stress in the rafter member is in the panel section adjacent to the heel of the truss and is equal to 35,000 lb. A resistance equal to 26,712 lb. is provided by the principal timber, so that a reenforcing piece provided through the two lower panels will be required to sustain a stress of 35,000 − 26,712 = 8,288 lb. Assuming that because of such practical considerations as notching for the framing of the struts, this timber must be at least 4 in. in depth, and as it must be of the same width as the main timber, its minimum size is 4 in. by 6 in., which is reduced by dressing to $3\frac{3}{4}$ in. by $5\frac{3}{4}$ in. The area of a timber of this size is 21.56 sq. in., so that if it has a safe unit stress of 400 lb., its total resistance will equal

8,624 lb., which is more than sufficient, since a stress of but 8,288 lb. must be resisted.

To determine whether the reenforcing piece possesses a safe resistance of at least 400 lb. per sq. in., it is necessary to apply formula **2**, taking for the value d, the dimension of the least side, $3\frac{3}{4}$ in. Substituting in this formula, the value u is found as follows: $u = 1{,}000 - \left(\frac{1{,}000}{100} \times \frac{110.64}{3.75}\right) = 705$ lb. The result of this calculation demonstrates that the reenforcing piece is much stronger than is required, though, for practical features of construction, it would not be advisable to use a timber of smaller size. Ans.

34. Although the method just described is usually of no practical value in saving material, it gives a simple means of making the last two strut joints, and when employed, the strength of the small pieces should be carefully figured by the principles illustrated. Frequently, owing to the unavailability of material of the proper size, the rafter member of a timber truss must be reenforced, in which case these principles for calculation may be advantageously applied.

35. Tension Members.—The size of a tension member is found by dividing the tensile stress to be resisted by the safe unit tensile strength of the material; the result will be the *net area* of timber required. The *gross area*, or the actual sectional area of the timber, must be one and one-half times the net area, in order to allow for cutting away in framing, notching, and boring for bolt and rod holes. When the gross sectional area of a timber used as a tension member has been found, the least dimension that may conveniently be assumed is divided into this, and the other dimension thus determined. It is customary in all truss construction to make the principal timbers of uniform width, in order that the framing at the joints may be simplified, and the face of the truss may be flush.

EXAMPLE.—A tension member is required to withstand a stress of 13,500 pounds. If the undressed width of the principal rafter member is 4 inches, what will be the required size of the tension member?

SOLUTION.—The allowable tensile resistance of commercial spruce is about 1,000 lb. The gross section required is $13{,}500 \div 1{,}000 = 13.5$ sq. in. One and one-half times this result equals 20.25 sq. in. Dividing this result by 4, the width of the member that is fixed by

consideration of the design, the calculated dimension is slightly more than 5 in. Since no timber is sawed whose dimensions are 4 in. by 5 in., the commercial size, 4 in. by 6 in., is adopted. A timber of this size could be dressed on all sides and still have ample section to resist the stress. Ans.

It is quite common, especially in small trusses, to make the compression and tension members of the same size, and they then possess a great excess of strength.

36. Laminated Compression and Tension Members.—It is sometimes advantageous to build a truss of thin planks bolted or lagscrewed together. But no matter how well the keying, bolting, etc. may be done, the action of the compression members must always, in conservative design, be considered as that of separate posts, and in calculating, the least dimension must be taken as the thickness of the plank used. The thickness of the planks composing the built-up member must in no instance be less than one-sixtieth of the unsupported length of the compression member. In building up a frame of planks bolted and screwed together, it is considered poor practice to use less than three planks, and tension members when thus built up should be so proportioned that two of the members are strong enough to withstand the entire stress, allowing for cutting, as in the example just given. The compression and tension members must be firmly fastened together with bolts or lagscrews, and in some instances keyed in order that the pieces may act in unison. Bolts for securing the timber should not be more than 3 feet apart, while when keys are deemed necessary they may be spaced 4 or 5 feet on centers.

CONNECTIONS

37. Joints in General.—It is always more difficult to secure the required resistance at the connections of a wooden frame than it is in iron or steel construction, since wood varies so much in its ability to resist shear and compression parallel with and perpendicular to the grain. Hence, in designing, great care must be taken not to exceed these various bearing values.

38. That the strength of a truss is as largely dependent on the strength of the joints as on the strength of the tension and compression members, is easily understood when it is considered that no member or structure is any stronger than its weakest point. If the joints are not designed with care and accuracy, the concentration of stress at these points will greatly endanger the stability of the structure. Therefore, in proportioning wooden trusses, a force acting at an angle with the grain should be resolved into two components, one parallel with and the other perpendicular to the grain, and the areas of the cuts in each particular case should be proportioned to the area found by dividing these components by the safe bearing value of the wood.

39. **Strut Joints.**—Although these joints are comparatively simple they are none the less important and should not be neglected. In designing **strut joints** it is desirable that the struts, or ties, be so arranged that their center lines intersect the center lines of the rafter and tie-members. Sometimes this is impossible, and in such cases the loads resting on the joints may be placed so that they counteract the rotation and consequent bending moment. The direction and size of the cuts should be carefully considered, although in many cases these cuts are made without properly considering the stresses produced, or the strength of the timbers resisting the stresses.

When a strut bears against a compression or tension member at any angle but a right angle, its stress should be resolved into two components each parallel to one of the cuts of the joint offering the resistance. Each of these components should, in turn, be resolved into two components, one parallel with and the other perpendicular to the grain of the strut, and each is also resolved into its components parallel with and perpendicular to the grain of the rafter member, and the areas may then be calculated for the safe stress according to the intensities of these components. Cases arise in which it is impossible to obtain sufficient area to resist the stress, in which instances the use of a cast-iron

or wrought-iron shoe will reduce the unit pressure to a safe limit. When metal ties extend through the cord members, washers must be used to reduce the stresses to their allowable limit.

40. There are two general types of strut joints, those connecting the strut to the upper chord and those connecting it to the lower chord. Although they are the same in principle, they differ slightly in their design.

A detail of the simplest form of strut joint is given in Fig. 11. Here the center lines of the strut, rafter member,

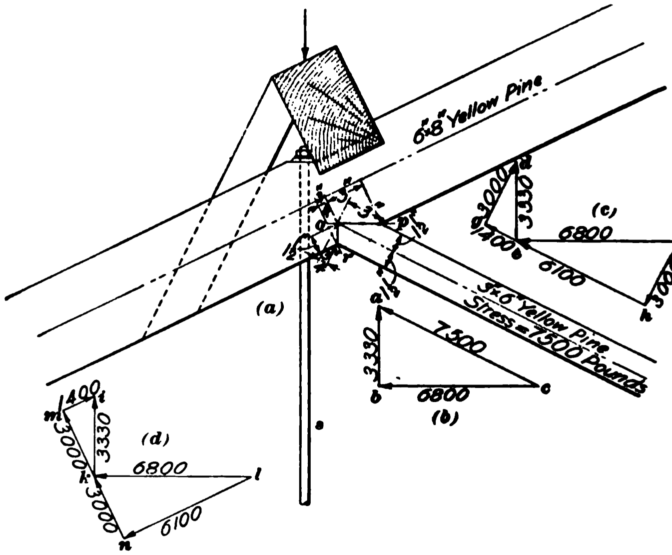


Fig. 11

and tie-rod intersect at a point corresponding with the panel point in the stress diagram. The joint is formed by notching the end of the strut into the compression member, the cut of the notch op being made to bisect the angle p between the strut and rafter member. This method of forming the cuts has the advantage of being convenient and usually disposes of the stresses to the best advantage. The principles and calculations involved in the design of this detail are illustrated in the following example:

EXAMPLE.—The stress in the strut shown in Fig. 11 is 7,500 pounds. Determine whether the cuts are large enough to reduce the stress per square inch so that the following allowable unit bearing values of the timber are not exceeded: Allowable unit compression on end grain, 2,000 pounds, and allowable unit compression perpendicular to the grain, 600 pounds.

SOLUTION.—In Fig. 11 (*b*), draw the line *a c* parallel to the strut, and lay off on this line, to some convenient scale, a stress of 7,500 lb. From the ends of this line draw *a b* and *c b* perpendicular to the cuts of the joints *o p* and *o r*, respectively. These components then represent the amount of stress that is taken by each of the cuts. In (*c*), to any scale, lay out these components from *d* to *e* and from *e* to *f*. Since the stress *d e* represents the amount of compression on the cut *o p*, by resolving this compression into its components parallel and perpendicular to the grain of the strut, it can be easily figured whether this surface is large enough. Draw *e g* parallel with the grain of the strut and *d g* perpendicular thereto. The component *d g* is found to be 3,000 lb. and is distributed across the projected area of the cut at the end of the strut, which, in this case, is 3 in. × 6 in. = 18 sq. in. The compression perpendicular to the grain is therefore $\frac{3000}{18}$, or 166 lb. per sq. in., which is much less than 600 lb., the allowable unit compression perpendicular to the grain.

The compression on this same cut parallel to the grain is 1,400 lb., as determined from *g e* in the diagram at (*c*), and the area resisting this is equal to the projected area of the cut on a plane at right angles to the stress from the strut and amounts to 6 in. × 1½ in. = 9 sq. in. The compression per square inch on this surface is therefore $\frac{1400}{9}$, or 155 lb., which is well within the allowable limit of 2,000 lb.

The cut *o r* of the strut in (*a*) has a direct stress of 6,800 lb. on it, and when this is resolved into its components parallel with and perpendicular to the grain of the strut, the stresses are found to be 6,100 lb. and 3,000 lb., respectively. The area resisting the compression perpendicular to the grain is ¾ in. × 6 in. = 4½ sq. in., and the compression per square inch is $\frac{3,000}{4\frac{1}{2}}$, or 666 lb., which is slightly in excess of the allowable limit, though not dangerously so. The compression parallel to the grain is 6,100 lb., and the area resisting this is 1½ in. × 6 in. = 9 sq. in. The compression per square inch is $\frac{6100}{9}$, or 677 lb., which is much less than the allowable bearing of the timber on the end wood.

In a similar manner the stresses on the rafter member are also found and the areas of the several cuts checked to determine whether the allowable bearing values are exceeded.

The diagram showing the intensity of these two components is shown in Fig. 11 (*d*) and is composed of forces of the same amounts as those in the diagram (*c*). It is evident that the projected areas of

that is, the compression on the joint *o r* paralle
and its area is 9 in., which is the same as alre
Hence, the joint is amply safe from failure by
in any direction, except that the stress for th
dicular to the grain of the joints is slightly gr
unit stress given in the example, but this exce
be disregarded in practice. Ans.

In the connection just analyzed, the
very little stress, its function being
chord of the truss. Hence, it is not c
its size, but to make it of either a $\frac{3}{4}$-inc
bar. In a truss no rods of less than $\frac{3}{4}$
be used. The ends of smaller rods
break off if the nuts are tightened too
the joints together.

41. A detail of a strut joint that m
tage where it is required to increase th
strut is given in Fig. 12. This design
is desired to cut the compression memb
In this case it is assumed that the cut i
may not be deeper than 1 inch. If the
in the figure, the compression perpendic
found, by the method in (*b*), to be 14,00
will require an area of $\frac{14000}{2000}$, or 7 sq
4″ × 4″ strut had been framed into the
required by the conditions of the prol
at (*c*), the area would have been too sm
fore introduced a cast-iron cap that

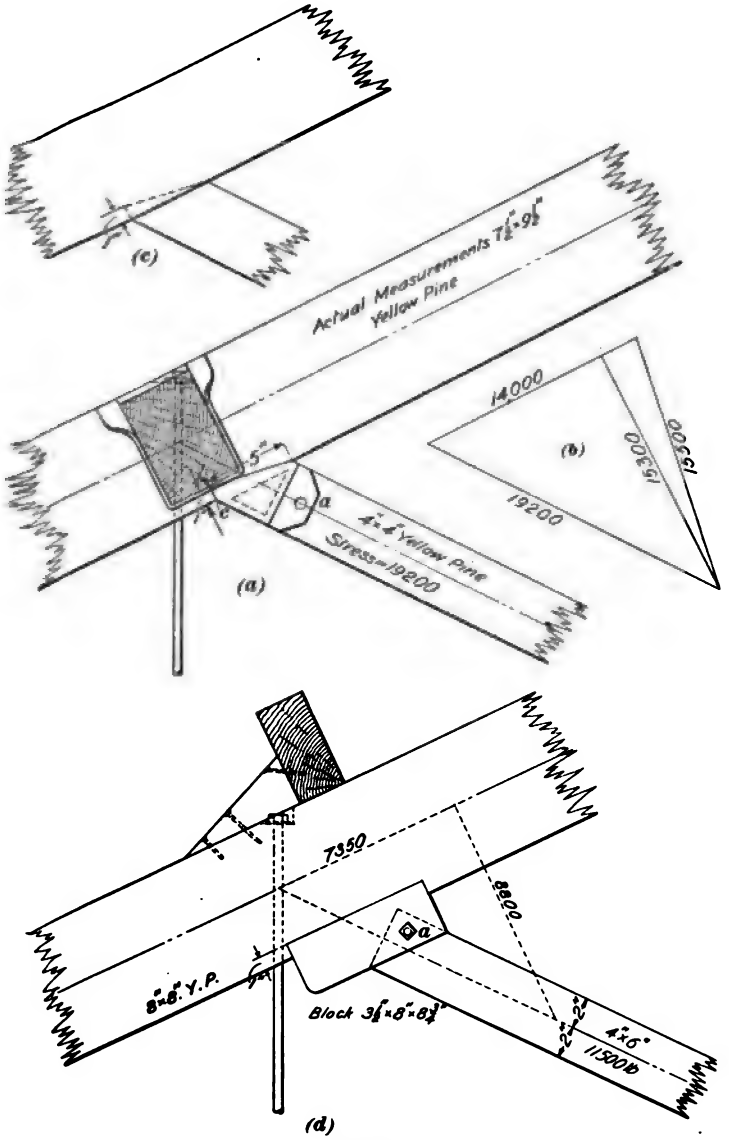

(c)
Actual Measurements 7½"×9½"
Yellow Pine
14000
15300
15600
(b)
19200
5"
a
4"×4" Yellow Pine
Stress=19200
(a)
7350
8800
a
8"×8" Y.P.
Block 3½"×8"×8½"
4"×6"
11500lb
(d)

FIG. 12

When stresses are light, no shoe is required, and the strut is simply notched into the rafter member, as in Fig. 12 (*c*), while in cases where a shoe is unavailable the strut may be framed into a white-oak block let into the rafter, as shown in Fig. 12 (*d*).

42. Fig. 13 illustrates a joint in which the stresses to be resisted are so great that a cast-iron shoe is required for the strut bearing. It is useful where the angle between the strut and compression member is small. The washer

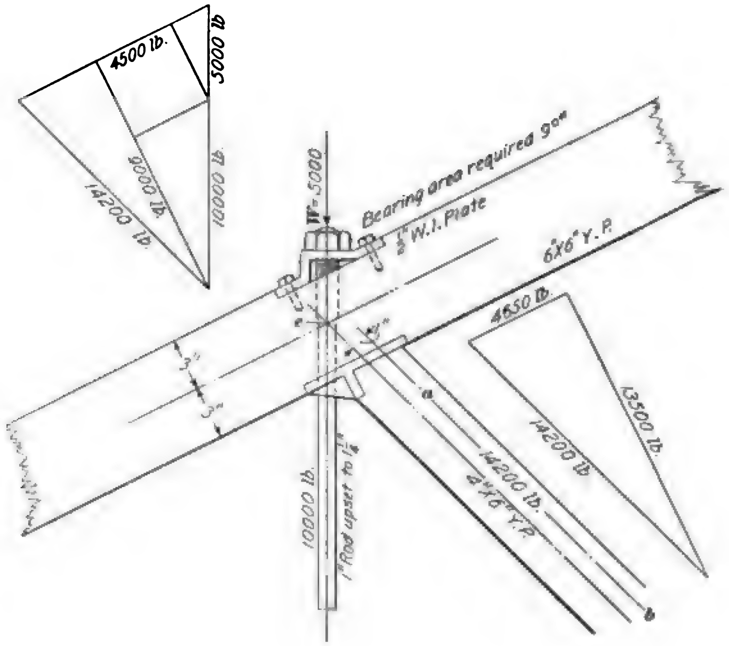


FIG. 13

shown is adapted to cases where, on account of the angle between the strut member and the rafter member, the stress parallel with the grain of the rafter member is reduced, and where it is desired to cut the compression member as little as possible. It is well to use lagscrews to hold the casting in place, but in many cases this precaution is not observed.

43. The strut joint given in Fig. 14 (*a*) is of advantage where the stresses are so great that in order to resist them, the end bearing of the strut must be increased. This is accomplished by introducing a cast-iron bearing plate that distributes the pressure of the strut over a large area of the upper chord member. This plate is provided with flanges cast on the sides between which the end of the strut member enters, and which serve as a guide and help to prevent this member from being displaced.

In this case very little cutting is necessary, as the strut or compression member is always perpendicular to the upper

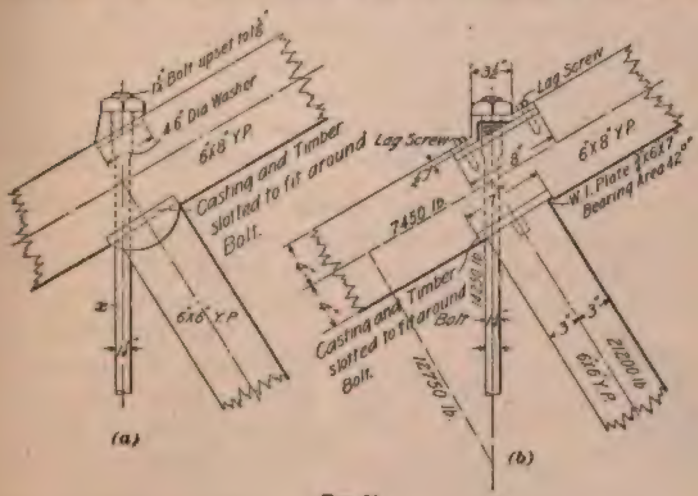


FIG. 14

chord. The tension member x is passed through a cast-iron washer of a design that is very useful where a number of trusses are to be built, as such washers are used throughout the upper chord. In such instances, it is advisable to proportion the bearing area of the washer for the tension member sustaining the greatest stress. The excess of area thus provided for the tension members of less stress is not objectionable, since but one pattern is required and complications resulting from the change of size are avoided. If the tension rods vary in diameter, the holes in the washers must be cored to suit.

In Fig. 14 (*b*) is given a method of forming a secure connection in which a wrought-iron plate is used instead of a casting, while in place of the side flange, a pin is employed to guide and hold the strut in position. Provision is made to resist the great stress by substituting a built-up steel-plate washer to supply the necessary bearing on the upper surface of the rafter member. It is not necessary to rivet the strap to the bent piece, for the lagscrew provides sufficient security against the tendency of the bent piece to spread. The holes through the straps for these lagscrews should be of such a size that the screws will fit them securely.

The detail shown in (*b*) may be used where large stresses are encountered. In many cases, especially where but few trusses are to be built, it is found cheaper and more convenient to have these washers made up than to use the cast-iron washers, as the saving on the cost of the patterns compensates for the cost of labor on the wrought-iron washers.

44. Another variety of cast-iron cap used for the connection of the strut to the compression member is shown in Fig. 15. Brackets are provided on each side to supply support for the purlins. The required resistance against sliding is obtained by setting the casting in the rafter member to the depth of the two projecting pieces *a*, *a*.

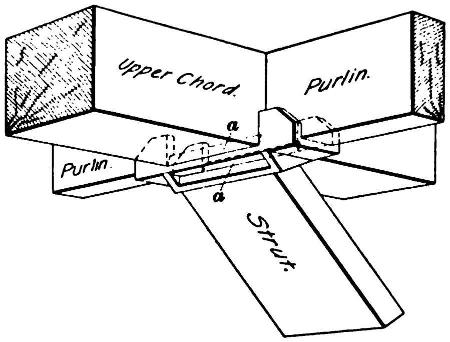


FIG. 15

45. All the details that have been shown were for trusses in which the struts slant and the ties are vertical. In the design given in Fig. 16 the struts are vertical and the ties are placed at an angle. The sliding component due to the stress in the tension member is taken up by the cast-iron washer, which is provided with the lip *a b* at the upper end. In designing this detail, care should be

taken that the area of the cut $a\,b$ is large enough to reduce the unit end compression on the rafter member to the safe stress, and that the shear along the line $a\,c$ does not exceed the allowable shearing stress of the timber. The bending moment on the metal at the point b should not be so great as to exceed the allowable unit transverse stress of cast iron, which is assumed to be 5,000 pounds. In this detail the vertical compression member is much smaller than the upper chord and instead of using any block or casting to connect it with the compression member, the strut has simply been let into the rafter member its full width.

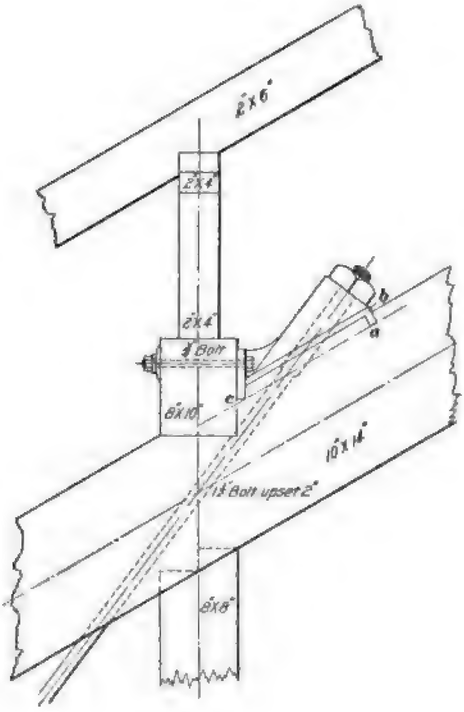


FIG. 16

46. Fig. 17 illustrates the connection between the lower end of a strut and the tension member of the truss. The same principles of construction are employed as in Fig. 11, and the joint is analyzed by means of similar diagrams and calculations.

47. Displaced Struts.—It is frequently desirable, especially in wooden trusses, to butt a strut against the upper chord or rafter member so that the center lines do not intersect at the panel points. As has been stated, this creates a tendency to rotate a portion of the chord about the panel point, causing a bending moment, which in many cases may

be balanced by placing the purlin s
and opposite moment around the
involved in the design of connection
misplaced are illustrated by the follo

EXAMPLE.—If the center line of the 4″ ×
in Fig. 13 had been so placed as to coincid
at what distance from its present positio
place the load, in order to balance the ben

SOLUTION.—Since this strut is 1 in.
moment produced is 14,200 × 1 = 14,200 in
by a load of 5,000 lb. placed on the truss a
equal to $\frac{14,200}{5,000}$, or 2.84 in., which is the
moved horizontally toward the right.

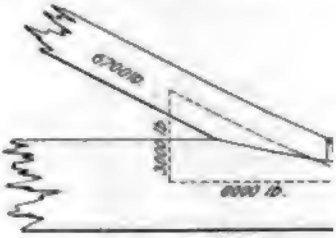


FIG. 17

48. Peak Joints.—A few of
peak joints are given in Fig. 18, t
detailed in (*a*), where the tension
two compression members and is l
washer. For light construction this

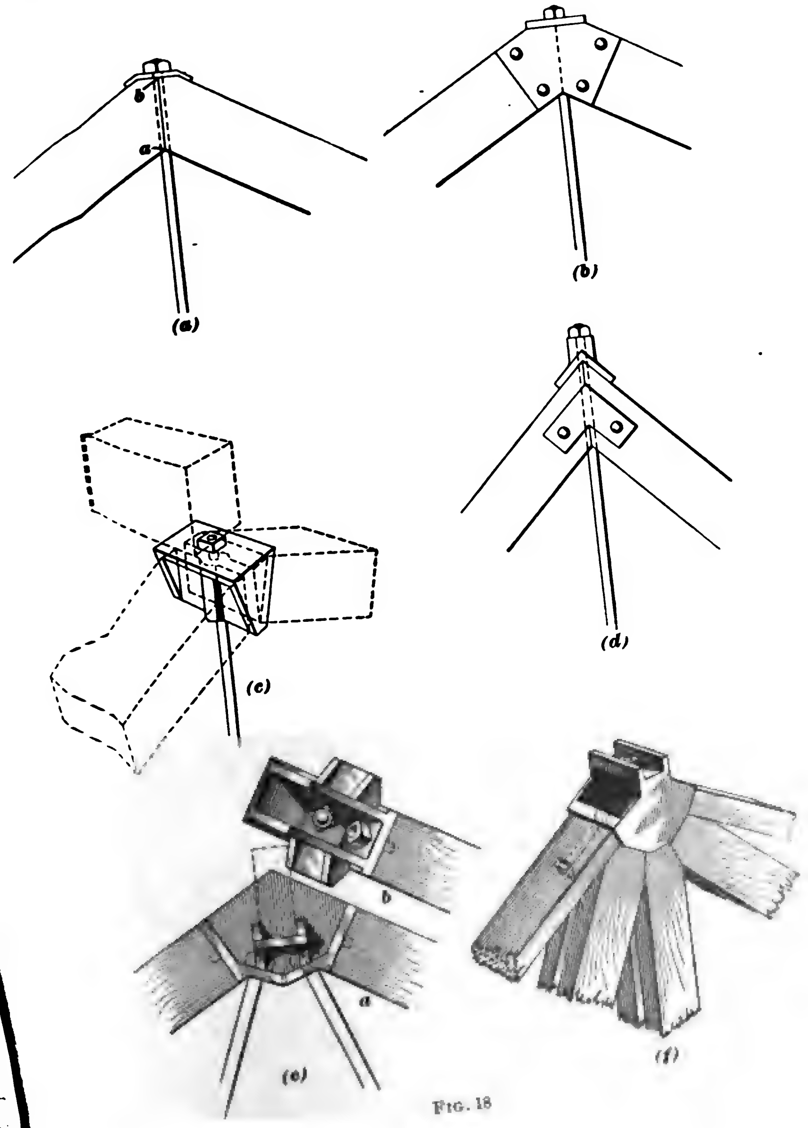


FIG. 18

cast-iron cap is introduced; this makes a very complete and efficient joint and is applicable to trusses in which the stresses are greater than would exist in (*a*) and (*b*).

The application of a cast-iron angle washer to a roof truss of steep pitch can be seen in (*d*), in which case the washer is made to fit the peak of the truss. If two tension members must be accommodated at this joint, a detail similar to the one in (*e*) may be adopted. On either side brackets are cast, into which the ridge pole is slipped, and a small hole is bored in the bottom of the casting to permit the escape of any water that might accidentally collect there.

The type in Fig. 18 (*f*) may be employed with advantage in cases where the stresses in the several members meeting at the peak are subjected to frequent changes by the wind shifting so as to affect first one side and then the other. The members coming together at the peaks are tension rods, or bolts, and wood compression pieces. The sides are left open to permit the tightening of the bolts throughout the connection. The top of the casting is raised and finished in the form of a bracket to form a support for the ridge pole. When the forms (*a*), (*b*), and (*d*) are adopted, a wrought-iron strap similar to the one illustrated in Fig. 19 may be employed to hold the ridge pole firmly in place.

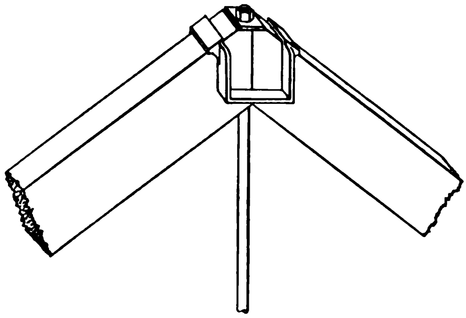
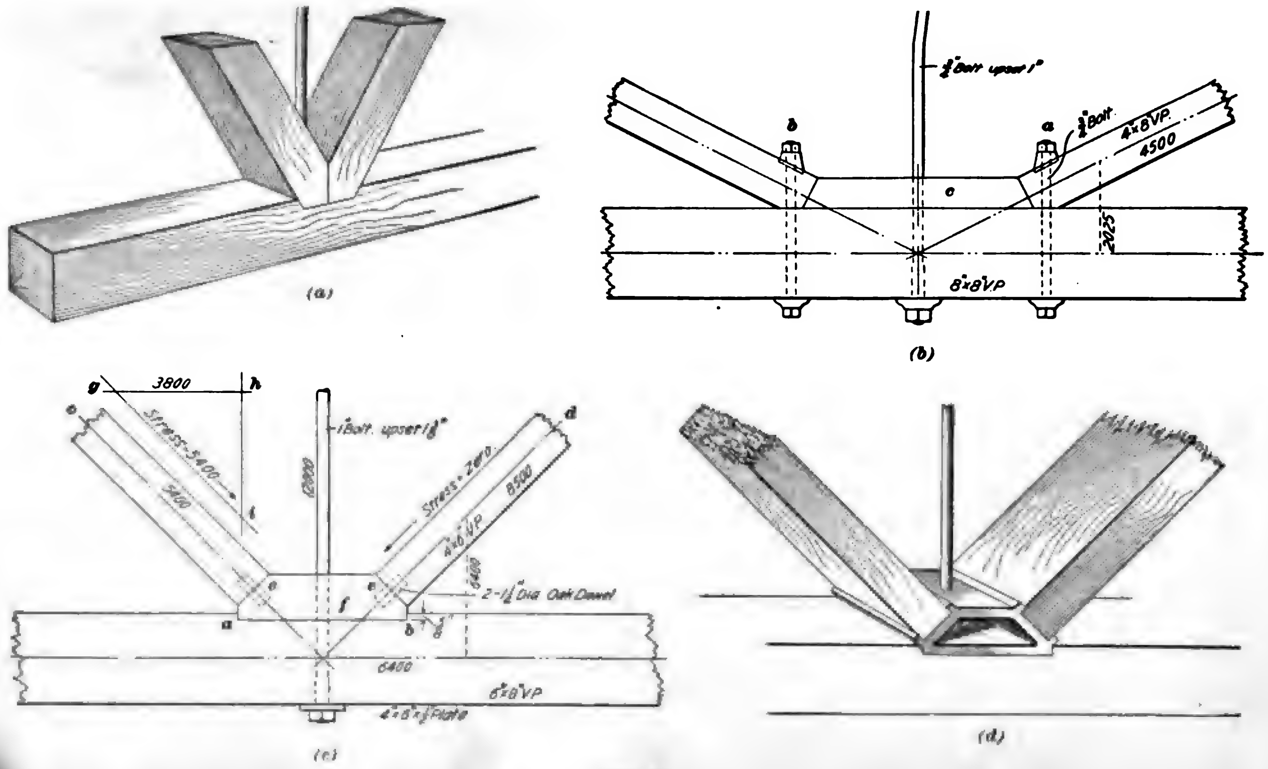


FIG. 19

49. Center Joints.—In Fig. 20 several types of **center joints** for wooden trusses are illustrated. In (*a*), the two struts butt against one another and are let into the tension member for a short distance, as a precaution against displacement. In (*b*), the angle formed by the struts and the tie-member is so small that a hardwood block *c* is inserted



(a)
b
a
3/4" Bolt upset 1"
3/4" Bolt
4"x8" V.P.
4500
c
2025
8"x8" V.P.
(b)
g
3800
h
Stress-5400
5400
1" Bolt upset 1 1/8"
12900
d
Stress-Zero
8500
4"x6" V.P.
6400
e
e
f
2-1 1/4" Dia. Oak Dowel
a
b
6400
6"x8" V.P.
4"x8"x1/2" Plate
(c)
(d)

Fig. 20

... the struts to hold them in position with- ... chord or tension member, and bolts are ... to make the joint more rigid.

... trusses subjected to any eccentric load, as a ... stresses are so variable that at a center joint ... meet and oppose each other, one must often ... greater stress than the other. When this varia- ... it is provided for by bolts a, b, Fig. 20 (b), ... considerable difference exists the joint or connec- ... detailed as in Fig. 20 (c). Here the struts ... in position by oak dowels e, e, and the block, ... which they bear is let into the tie-member. ... figure to what depth the tension member must ... receive the block f, the difference in stress on the ... should be determined from the wind-load diagram. ... held in place by the thrust of the struts and the ... although it is very important that the cuts at ... with the greatest care in order to secure a tight ... may, in many cases, be more readily accomplished ... these cuts with a slight taper. The necessity of ... between the block and the tie-member is obvious. ... stress would cause a slight play in the ... to rock back and forth, which would be ... the stiffness of the truss.

... figuring the area of the cut for the block is ... following example:

... the stress due to the wind load in the ... when the stress from the wind is zero in d; ... figure supply sufficient bearing area at b, pro- ... of the timber in end bearing is 1,600 pounds?

... triangle of forces at $g\ h\ i$, resolving the stress ... horizontal and vertical components. From this it ... unbalanced horizontal force of 3,800 lb. ... to the right. The area of the cut must ... or 2.375 sq. in. Since the width of the ... depth is $\frac{2.37}{6}$, or about .4 in. The actual ... or .625 in., so that the distance the ... will be ample.

In the type shown in Fig. 20 (*d*), a cast-iron block is used, and the struts are held in place by projecting lips made on the casting.

51. Heel Joints for Light Loads.—More difficulty is experienced in designing the **heel joint** than any other, for the reason that in a truss the stresses accumulate until the heel is reached, and are greatest at that point. On this account the heel is the most important joint, and, in timber trusses, usually requires careful study in order to secure the necessary resistance.

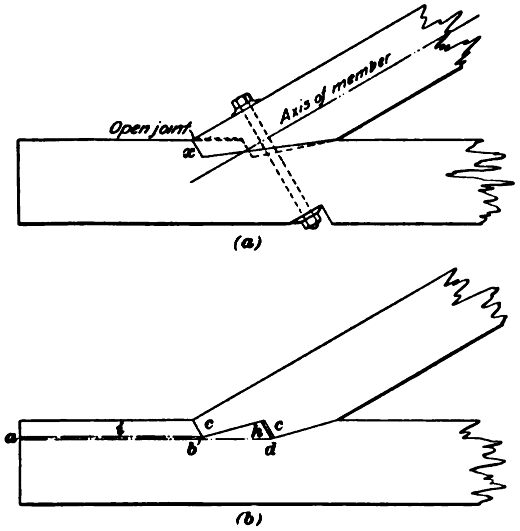


FIG. 21

The simplest type of heel joint, shown in Fig. 21 (*a*), is adapted to trusses in which the stresses are small. The portion of the tension members projecting beyond the notch has sufficient shearing resistance to take all the horizontal pull in the lower chord, while a bolt holds the members in position. An objection might be raised that the area x, which resists the thrust of the rafter, is not concentric with the line of stress, which is considered as coinciding with the axis of the member. A suggested improvement is

indicated by the dotted lines, in which the resisting area is made concentric with the thrust of the upper chord. In order that the thrust of the rafter may not have a tendency to break off the toe of the member, the end is left with an open joint, as shown.

It is seldom advisable to make two cuts, as in (*b*), because through careless workmanship and by shrinkage, one of the bearings at *c*, *c* is sure to become inactive, leaving the other to withstand all the stress, so that the shear along either *a d* or *a b* only will be realized and the pieces *h* and *i* will shear off successively instead of conjunctively, greatly weakening the joint.

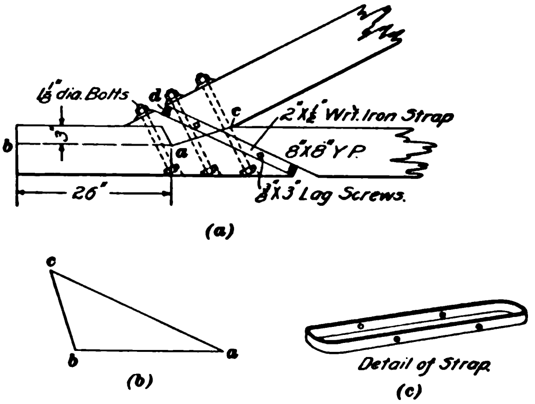


FIG. 22

52. When the shearing strength of the projecting member is not sufficient to resist the thrust of the upper chord, the design shown in Fig. 22 may be adopted. The extra strength required is supplied by the use of a wrought-iron strap, the bolts serving to make a more rigid connection, although they are not considered when figuring the resistance of the joint. Such a joint should be used only when well-seasoned timber can be secured. The calculations for determining the strength of the joint are given in the following example:

EXAMPLE.—If, in a joint constructed as in Fig. 22, the stress in the rafter member is 39,000 pounds and in the tension member, 37,300 pounds, what size wrought-iron strap will be required, providing

he dimensions of the timber and the connections are the same as hose given in the figure, and the allowable unit shearing resistance of the timber parallel with the grain is considered as 120 pounds, and he allowable unit tensile stress of the strap is taken as 16,000 pounds?

SOLUTION.—Since the length from *a* to *b* is 26 in. and the width of the piece is 8 in., the area resisting the thrust is 26 in. by 8 in. = 208 sq. in., which multiplied by 120 lb. gives 208 × 120 = 24,960 lb., shearing strength. Then the remaining stress to be taken care of by the strap is 37,300 lb. − 24,960 lb. = 12,340 lb.; for convenience, say 12,400 lb. In the diagram shown in (*b*), lay off, to some convenient scale, *a b* equal to 12,400 lb. This will be one component of the stress in the strap. The other component exists as a pressure on the cut *a c* of the connection, as shown in (*a*). In the diagram at (*b*) draw *a c* parallel with the center line of the strap shown in (*a*), and *b c* perpendicular to *a c* in (*a*); this locates the point *c*. Then, by the same scale, measure the length of the line *a c*. This length represents the stress that the strap will be required to resist, and is found to equal 15,750 lb. As the strap is 2 in. by ½ in., the area on both sides is 2 sq. in., so that if a safe unit fiber stress of 15,000 lb. is assumed, the strap will have a tensile resistance of 30,000 lb. Thus, it is observed that the strap has a resistance nearly double that actually required; although, on account of the fact that if the strap were made narrower, it would, under the full stress, be likely to crush or cut the timber at *d*, Fig. 22 (*a*), its width cannot well be reduced. Besides, the strap must be welded in at least one place, and it must be drilled for the lagscrew, which will reduce its gross sectional area to about 1.563. These facts, when considered, lead to the conclusion that the strap as designed is none too large, for it is not safe to figure on more than 75 per cent. of the strength of the section where a weld occurs. Ans.

53. In Fig. 23 is given a method for framing the heel joint of a truss with a steep pitch. A cast-iron shoe with a lip *a* is employed. The strength of the joint depends on the strength of the lip and the shearing strength of the tension member *b* along the line *c d*. In designing this connection no reliance is placed on the bolts and lagscrews.

The figure also illustrates a possible construction for framing the back wall and roof. Here a 3″ × 12″ wall plate is bolted to the end of the tie-member *b*, and supports the rafter along its length, the brickwork being carried up on each side of the truss to the under side of the plate. The ends of the rafters are covered and protected by a galvanized-iron cornice *e*.

54. In Fig. 24 is shown an excellent detail adaptable to trusses of intermediate spans, or spans of from 30 to 50 feet. The special advantage of joints of this type is that a strap that can be so adjusted as to be drawn tight and thus take up any shrinkage that may occur in the timber is used. The design has the further advantage that no bolt holes need be bored in the timber, thereby effecting a saving, and avoiding the necessity of cutting the members.

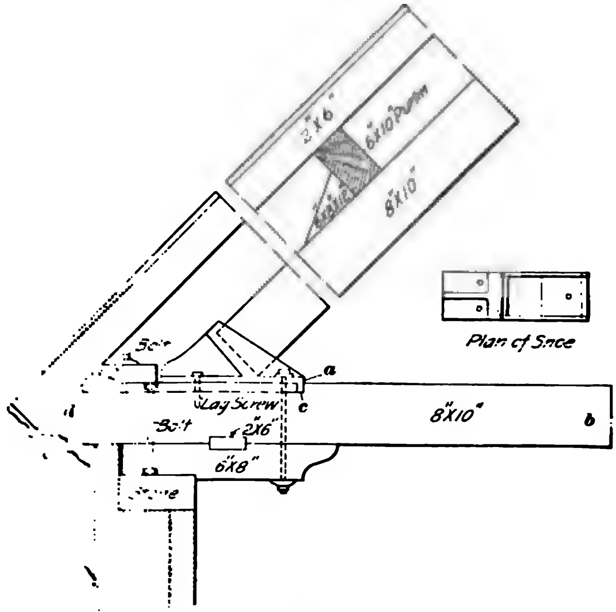


FIG. 23

[illegible] tendency of the strap to crush the corner of the tim- [illegible] which it passes is overcome by placing a cast- [illegible] the upper chord, while a casting *c* shaped so as [illegible] bearing for the nuts is used at the lower end. [illegible] construction are clearly designated in that [illegible] elevation showing a section on the line *x x*. [illegible] the lower chord of the truss projects far [illegible] of the rafter member to give sufficient [illegible] the whole thrust, and for this reason a rod

$\frac{3}{4}$ inch in diameter is sufficient to hold the joint in position. But if the rod is proportioned to stand the whole stress, it must have a diameter of $1\frac{1}{4}$ inches at its weakest section, which occurs at the root of the thread. The calculations for the strength of this strap are similar to those given for Fig. 22.

55. Laminated Truss Heel Joints.—The constructions shown in Fig. 25 are usually adopted when the tension and compression members are made of plank. Although some authorities claim that bolts should never be used as pins, since the length is so great in proportion to the

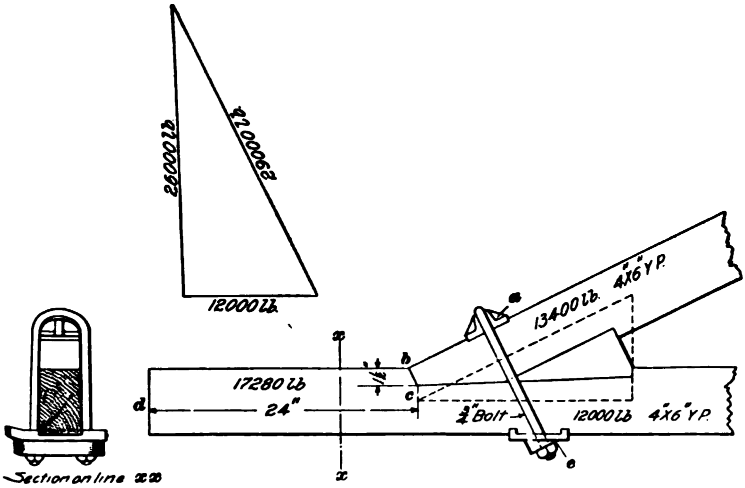


Fig. 24

diameter that the bending moment is excessive, they may be employed with advantage in cases where the stresses are not very great. In the present case the bolts have an effective span of 5 inches, as shown in the end view, and if the joint is designed as in (*a*), the bolts must be proportioned to stand the full stress of the compression members.

If the bolts are considered as resisting equal shares of the stress from the oblique or compression members, each is assumed to support a concentrated load of $\frac{2{,}225 + 2{,}225}{4}$ = 1,112 pounds. From this load each bolt will be subjected

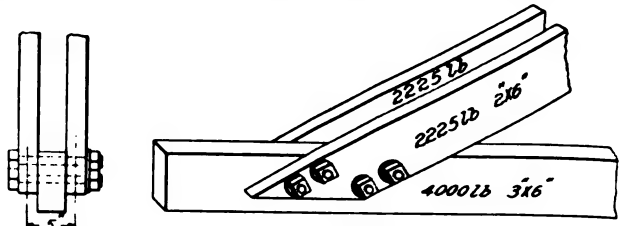
to a bending moment of 1,390 inch-pounds, which is calculated by applying the formula

$$M = \frac{WL}{4} \qquad (4)$$

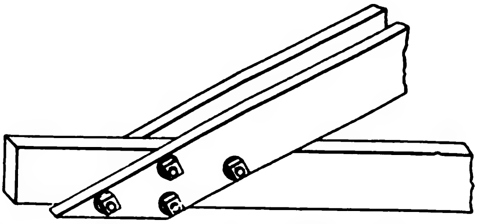
in which W = concentrated load of 1,112 pounds;

L = distance between centers of supports, or 5 inches.

To resist this bending moment there will be required a section modulus equal to $\frac{1390}{12000}$, or .116 for each bolt, provided



(a)


(b)

modulus of a 1-inch bolt is .0982 and that of a $1\frac{1}{8}$-inch bolt is about .139. The former, though somewhat light, can be used with safety.

If the compression member is carried down, as shown in Fig. 25 (*b*), so that it rests directly on the bearing block, the vertical component of this stress is balanced, and the only stress to be resisted by the bolts is the direct pull in the tension member. In this instance the required section modulus for one bolt is only .104, which shows a saving over the former construction, or at least a greater margin of safety.

If the tension member had been let into the compression member a short distance, as in (*c*), the bending moment on

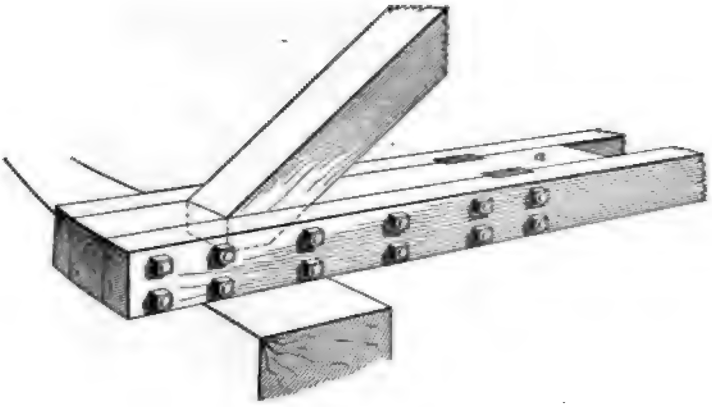


FIG. 26

the bolt would have been entirely eliminated and the only office performed by the bolts would be to hold the parts together.

In order to secure alinement of the holes they should be bored when the pieces are in position, the work being done very carefully in order to secure a snug fit for the four $\frac{3}{4}$-inch bolts that are required for a joint of this design.

56. Fig. 26 illustrates a detail applicable to trusses made up of small members. To avoid cutting the tension pieces from the top surface, a block *a* is inserted between the sides of the two tie-members and secured to them by keys and

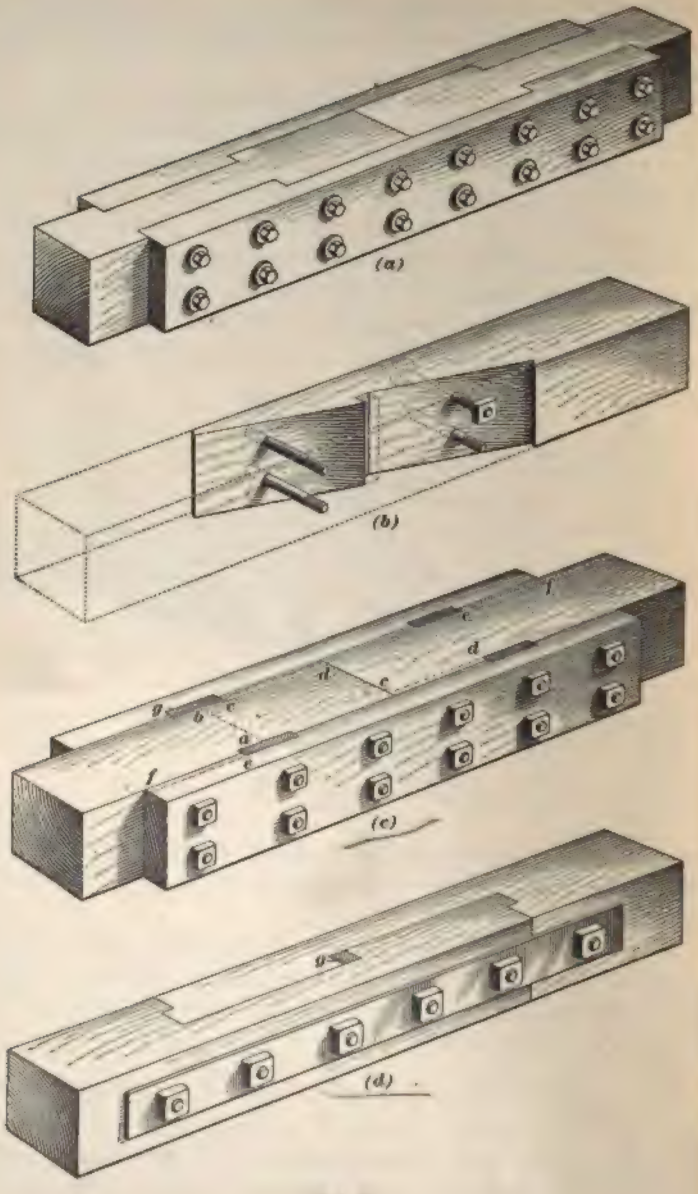


FIG. 27

bolts. The compression member is let into this block to a sufficient depth to avoid any possibility of inadequate bearing, while its load or stress is depended on to prevent its being displaced.

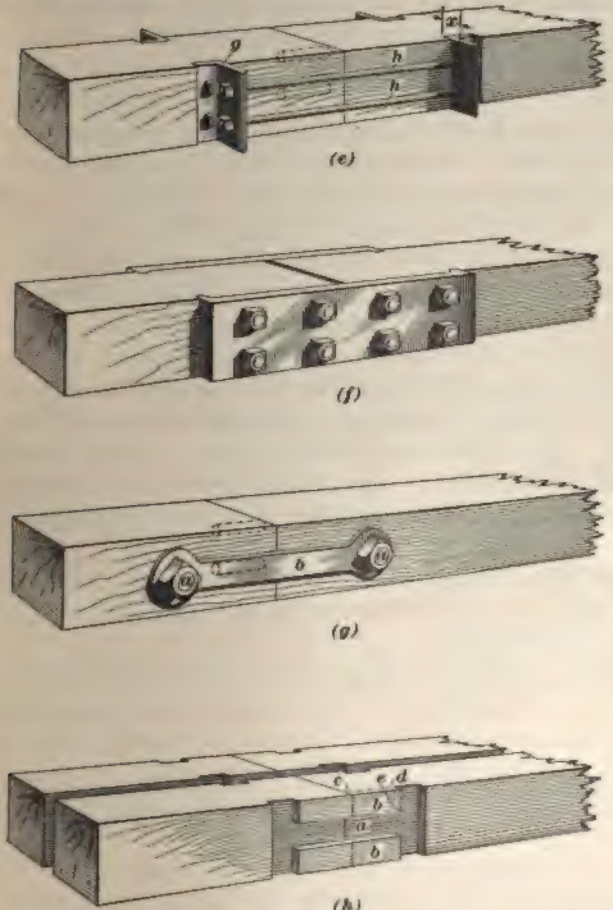


Fig. 27

57. Splices.—A number of splice joints for wooden tension members are illustrated in Fig. 27, the one in (*a*) being formed by butting two pieces together and using two splice plates of wood that are let into two opposite sides of

the tension members, and held in position by means of bolts. The splice in (*b*) is varied by being cut obliquely and toothed, and is tightened by the use of keys and held together by bolts. This is very convenient in some cases, but is not as strong as the type in (*c*). In (*d*), the cut is made lengthwise of the piece, and the joint is strengthened by means of metal bars; this form may be used in trusses that have a great deal of stress coming on the connection. In the splices in (*e*), (*f*), and (*g*), metal is employed to supply the required strength and pins are used to guide the members and keep them in line. In (*f*), the ends of the cast-iron plates are made with a slight bevel, so that in tightening the bolts the two parts of the tension member are drawn together. When the truss under consideration is of laminated construction, such a detail as the one shown in (*h*) forms a simple and effective splice; this design is especially useful where it is desired that the full width of the splice shall not exceed the width of the tension piece. A wrought-iron plate is cut to the form shown at *a* and set into the wood. One such plate is used on each side, being held in place by bolts or lagscrews. The principles involved in determining the strength of a splice joint or connection of a tension or tie-member are the same as those explained in connection with the detail design of heel and strut connections.

In the design of all the splice connections shown in Fig. 27, after the maximum stress in the member has been obtained from the stress diagram, it is necessary to investigate the joint usually for failure in three ways. For instance, referring to the joint in Fig. 27 (*c*), it is first necessary to determine whether there is sufficient sectional area of timber on the line *a b* to resist the stress in the tie-member; then it must be decided whether the timber used for the tie-member supplies sufficient resistance to shearing along the line *c d*, and third, the wood splice plates along the line *e f* must be considered in order that it may be known whether there is sufficient resistance to supply the necessary stress. It is also well to investigate the bearing on the end wood of such

-faces as those marked g, g, Fig. 27 (c), (d), and (e); also, cases where iron is used, as in (e), the bolts h, h in tension ıst be proportioned to withstand the entire stress to which e member is subjected, and in determining the strength, e area at the root of the thread is to be taken, unless the ›lts are *upset*, or enlarged on the ends so that the area the root of the thread will be at least 10 per cent. greater ıan the area of the body of the bolt.

The angles are subjected to bending from the pull of the olts, the bending moment being equal to the tension in the olts h, h multiplied by the distance x. The resistance that he angles offer to this bending stress should be calculated, ınd if the resistance is less than required, angles of greater veight must be used.

The detail in Fig. 27 (g) is weak from the fact that the bolts a, a bear directly against the wood, and are apt to crush or split the wood before their full strength can be realized. A better arrangement is provided by setting the bar b into the timber, driving it tightly in place to make it fit well. As a rule, the resistance of the bolts in splice connections should be neglected, except in such constructions as in (g). In this case the bolt, which partakes more of the nature of a pin, should be of considerable size and analyzed for bending.

The principal points for consideration in the detail in Fig. 27 (h) are whether the pieces b, b have sufficient shearing resistance along the line $c d$ and if there is enough bearing provided at $e d$ on the end wood to realize a resistance equal to the stress.

58. An example illustrating the calculations necessary in determining the strength of a splice joint is given in the following:

EXAMPLE.—In Fig. 28 is shown the splice of a tie-beam in a wooden roof truss composed of yellow pine. What is the strength of the splice, disregarding the bolts a, a entirely?

SOLUTION.—The strength of the splice depends on the tensile strength of the wood at the net section $e f$ and on the tensile strength of the net section of the two splice plates. It also depends on the tendency of the splice plates to shear along the lines $s t$ and $s' t'$, and on

the tendency of the tie to shear along the lines *m n* and *m' n'*. Assume the areas of the net sections to be sufficient to make their strength greater than that of the sections that will fail by shearing; then, referring to Fig. 28, it will be seen that the line of shear on the splice plate at *s t* and *s' t'* is longer than that of the tie-member at *m n* and *m' n'*; therefore, in computing the strength of the splice, the strength of the

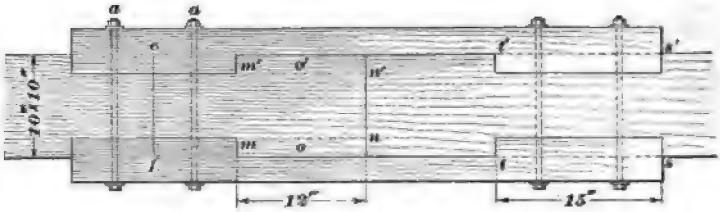


FIG. 28

tie need be considered along the lines *m n* and *m' n'* only. The pieces *o*, *o'* tend to slide or shear off from the main tie along the lines *m n* and *m' n'*. The area along these lines is 12 × 10 × 2 = 240 sq. in.; and since the ultimate shear of yellow pine parallel to the grain is 400 lb. per sq. in., the ultimate strength of the splice, disregarding the bolts *a*, *a*, is 240 × 400 = 96,000 lb. Ans.

59. Heel Joints in Trusses of Long Spans.—The detail given in Fig. 29 may be employed in cases where the stresses at the heel joint are great, and the projection of the tension member beyond the support or bearing of the rafter member is limited. To reenforce the tension member where the lip of the shoe cuts into it, a 6″ × 6″ bolster is introduced and is connected to the tension member by means of the ¾-inch bolts shown at *a*, *a*. The tendency of the tie-member or tension member to slide along the bolster *b* is resisted by keys *k*, *k* let into the bolster and tension member. The wall bearing is located directly under the junction of the center lines of the several members so that the tie-member is subjected to no bending stress, as it would be if this intersection were located outside the edge of the wall. The ⅞-inch bolt *g* overcomes any tendency of the cast-iron shoe *c* to slip out of the cut provided for its lip *d*.

60. The type of heel connection illustrated in Fig. 30 may be effectively used in timber trusses of large span.

ıe bolts introduced in this detail greatly increase the rength of the joint. The calculations necessary for the esign are as follows:

The outward thrust of the compression member, which is qual to the pull of the tension member, or 32,250 pounds, resisted by the shear of the lower chord along the plane bc, nd by the horizontal component of the tension in the olts d, d. The allowable, or safe, shear of the wood along he plane bc is first calculated, and the bolts then propor-

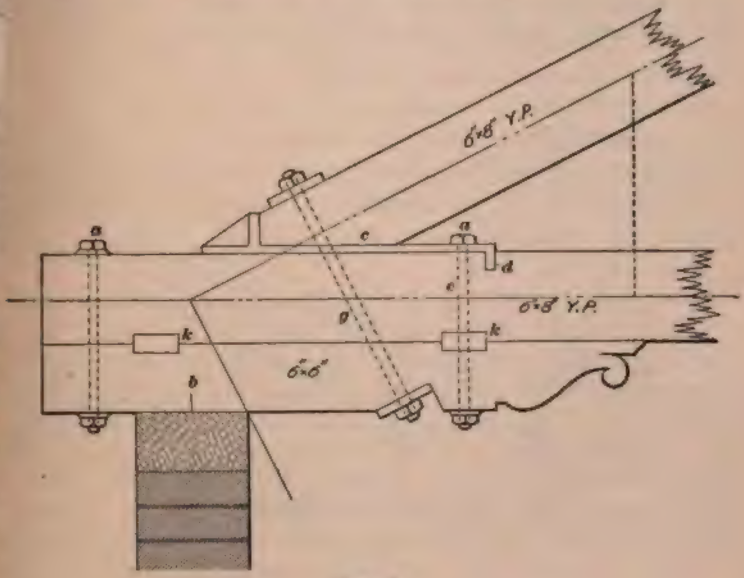


FIG. 29

tioned to resist the remaining stress by their horizontal components. The shearing strength of the wood along the line bc, when 120 pounds is taken as the allowable unit stress, is $120 \times 6 \times 29\frac{1}{2} = 21{,}240$ pounds, leaving a remaining stress of 11,010 pounds to be sustained by the bolts. Since the bolts are placed perpendicular to the upper edge of the rafter member, and the horizontal component of their stress is known, the triangle of forces $c' a' b$ may be drawn by laying off $c' d'$ in a horizontal direction equal to 11,010

pounds and drawing $c' b'$ vertically, and $d' b'$ parallel in direction with the center line of the bolts. The line $d' b'$, which represents the amount of stress to be resisted by the bolts, scales 24,800 pounds. If it is considered that the bolts are of first-class rolled bar iron and have a safe unit tensile resistance of 18,000 pounds, then the combined net area of the bolts will need to equal $\frac{24800}{18000}$, or 1.37 square inches. The area of the root of the thread of a 1⅛-inch bolt is .694 square inch, from which it is evident that as two such bolts will have a combined area at the root of the thread of 1.388 square inches, they will be sufficient. In order to

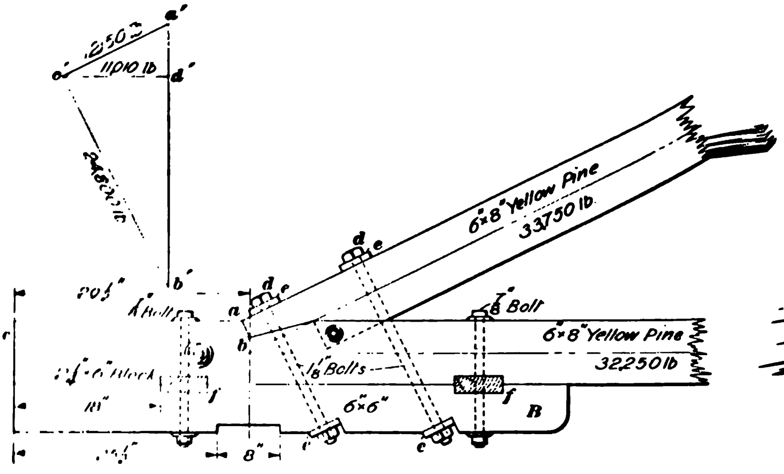


FIG. 30

obtain enough bearing area, so that the allowable unit bearing value of the timber perpendicular to the grain is not exceeded, the washers at e, e should extend across the entire width of the members. Georgia yellow pine, of which this truss is assumed to be constructed, has an allowable bearing, perpendicular to the grain, of about 600 pounds, so that if the pressure is 24,800 pounds, and the length of the bearing, or width of the timber, is 8 inches, the width of the washer will be $\frac{24800}{600 \times 8} = 5.16$ inches. Hence, a washer made of a

piece of 5-inch bar iron 8 inches long and having two 1¼-inch holes bored in it would be practical. Such washers are usually made ¾ inch to 1 inch in thickness.

61. The bolts in this construction are carried through the tension member and the bolster *B*, which is used to avoid cutting the tension member for the bolts *d*, *d*. The horizontal component of the stress in the bolts is transmitted from the bolster to the tension members by means of blocks *f*, *f*. This horizontal component is 11,010 pounds; hence, if one of the blocks is located 18 inches from the end of lower chord member, the shearing strength of the wood parallel with the grain will be $18 \times 6 \times 120 = 12,960$ pounds, which will be sufficient.

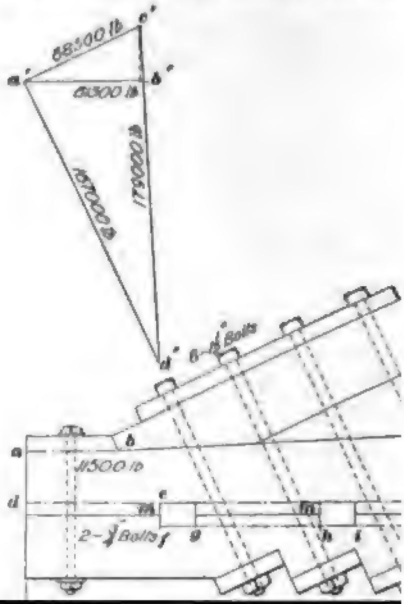
The depth, or thickness, of the oak blocks is determined by the bearing area required at the edge of the block. Considering the allowable unit bearing of oak perpendicular to the grain as 700 pounds, the depth that the block should cut into either the tie or bolster should be $\frac{11,010}{700 \times 8 \times 2} = .98$ inch, and the block should be made 2 inches in thickness or depth.

Besides stress investigation the width of the oak block should be analyzed to see whether it has sufficient shearing resistance in a horizontal plane to withstand a lateral stress of 11,010 pounds.

62. In Figs. 31 and 32 are shown two types of heel connection for trusses in which the stresses are very great. It is assumed that the design of the building limits the projection of the lower chord member to that shown in the figure. Since the stresses are high, the bolts must be designed to supply the strength of the joint.

In Fig. 31, the shearing strength on the plane *a b* is 11,500 pounds, leaving 73,000 − 11,500, or 61,500 pounds, to be taken by the bolts. In the stress diagram lay off *a′ b′*, representing 61,500 pounds, and draw a vertical line through *b′*, prolonging it until it intersects a line drawn through *a′* parallel to the upper chord, at some point *c′*. The amount of stress in the upper chord to be resisted by the bolts is obtained by measuring the line *a′ c′* with the scale employed in laying off *a′ b′*.

This stress is resisted by a dir
and by the tensile resistance of t
bolts is therefore found by drawir
to the cut $b\,c$, and the line $a'\,d''$ p
bolts; then the length of the line
to be resisted by the bolts, or 167
are used, it is found by calculat
given, that their diameter should



be at least $\frac{61{,}500}{8 \times 200} = 38$ inches. As the design does not permit the use of keys of this thickness, resort must be had to keys made of either cast or wrought iron, and whose depth must be such that the pressure of the surfaces at m, m, m does not exceed the allowable bearing value of the timbers on end wood. In Fig. 31 the section represented by the dotted lines withstands the shearing stress, and a portion of the length of the bolster is useless. The length

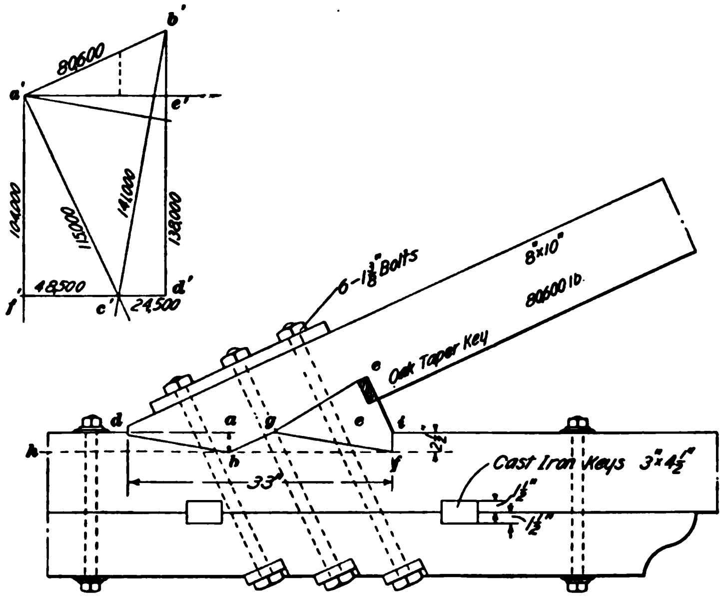


Fig. 32

of the bolster is equal to the sum of the widths of the keys plus the length of the portions of the tension member or tie-member in shear, that is, gh, ij, and kl plus the projecting end that is not in shear and is made equal to de. If these distances are made of equal length, their sum plus the width of the keys, or the length of the bolster, may be expressed by the formula

$$L = a + b\left(\frac{n+1}{n}\right) \qquad (5)$$

o = required length for resist
n = number of keys employed

Applying the formula, the required le using three cast-iron keys each $4\frac{1}{2}$ $13\frac{1}{2} + 63\left(\frac{3+1}{3}\right) = 97\frac{1}{2}$ inches. In t length of the bolster shown is 7 feet 3 i if this length is advanced to the amou factor of safety will be reduced, for the must be decreased. The width of the smaller, but in that case they would te the notches. In instances of this kind decide whether the length of the bolste made as required.

If the entire stress of 61,500 pounc uted among three keys, and the allow 2,000 pounds, the area required for keys against the end grain of the tin is $\frac{61,500}{2,000 \times 3} = 10.25$ inches. The wic 8 inches, so that the distance which t the wood is $10.25 \div 8$, or about $1\frac{1}{4}$ inc the key will, in consequence, equal $2\frac{1}{2}$

63. The method just given is quite in framing heavy joints, but it is not esp because the whole tendency of the joint

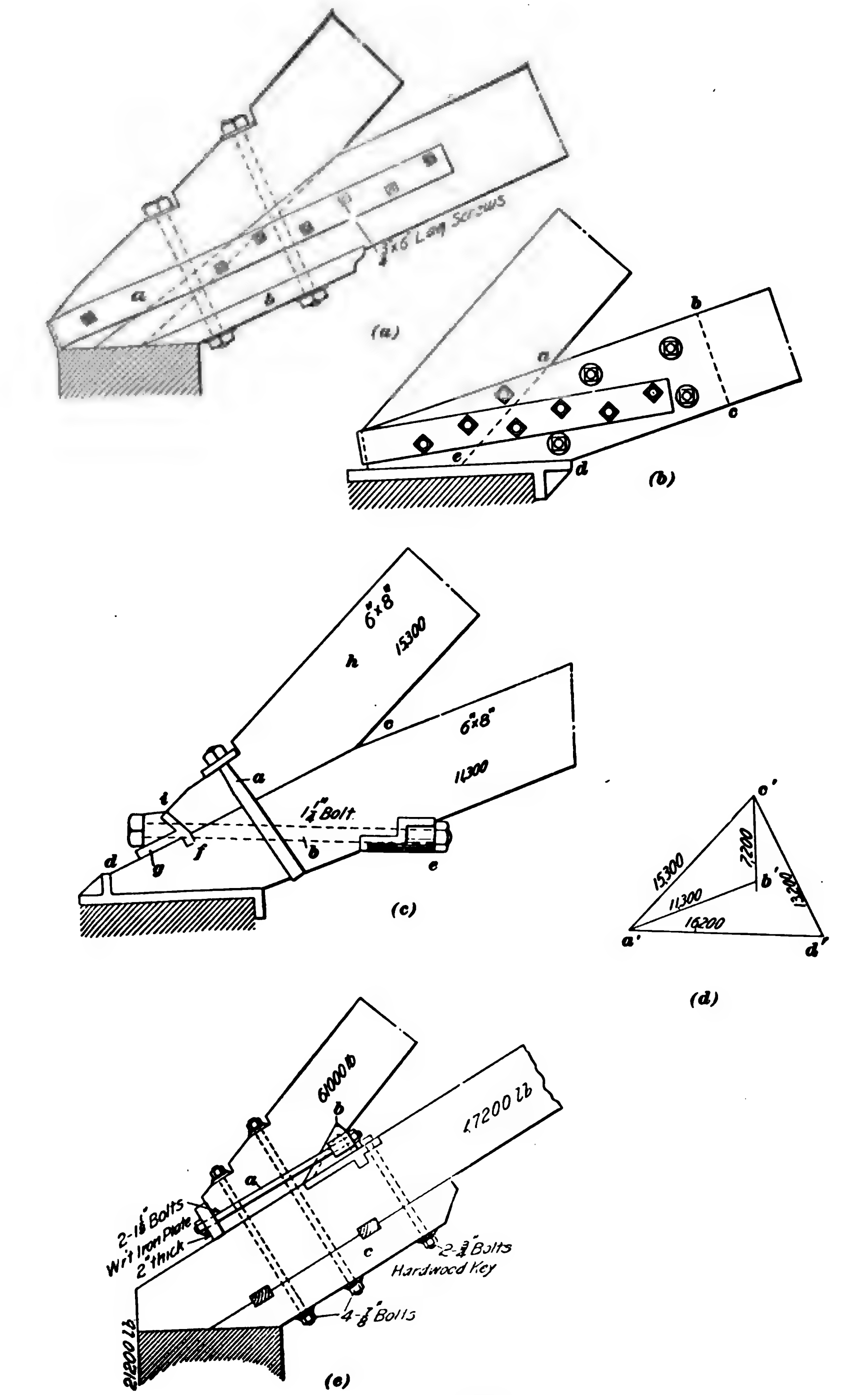
parallel to the face $b\,d$ is inserted at c. The tendency of the joint to slide upwards is resisted by the bolts.

In order to determine the stresses in the several structural elements, the stress diagram is laid off, in which the distance $a'\,b'$ represents the compressive stress in the oblique member. Draw the line $b'\,c'$ perpendicular to the direction of the cut $d\,b$, and $a'\,c'$ parallel to the direction of the bolts. Then the length of $a'\,c'$ represents the stress in the bolts, and $b'\,c'$ the pressure on the cuts $d\,b$ and $g\,f$. The required resistance of the bolts is found to be 115,000 pounds, and hence six $1\frac{3}{8}$-inch bolts will be required to resist this stress. The pressure on the tension member perpendicular to the grain is 138,000 pounds, and if the allowable unit resistance is 600 pounds, the bearing area required is $\frac{138000}{600} = 230$ square inches. The total length of the cut is 33 inches, and its width 8 inches, giving an area of 264 square inches, which is in excess of the area required, so that it is more than safe. The stress of 24,500 pounds parallel to the grain is resisted by a surface equal to the sum of the vertical projections of the cuts $g\,b\,d$ and $i\,f\,g$, or 5 inches. This measurement, multiplied by the width of the timber, will give sufficient area to resist the pressure on the end grain of the tie-member or chord. From the fact that no cuts are made by which direct resistance is applied to the thrust of the upper chord or oblique member, the whole compression in this chord should be considered as above in the calculations for the bolts. The cuts for the keys must then be of such proportions as to resist the whole of the horizontal component of the stress in the bolt. This force, which tends to slide the bolster along the lower chord, is 48,500 pounds, as determined by scale from the line $f'\,c'$ in the stress diagram.

Applying the formula for two keys, the tie-member being of Georgia yellow pine with an assumed allowable unit shearing resistance parallel with the grain of 120 pounds, $L = 9 + \frac{48,500}{8 \times 120}\left(\frac{2+1}{2}\right)$, or $L = 9 + 50.52\left(\frac{2+1}{2}\right) = 84.78$ inches. The depth of the keys can be found by dividing the entire horizontal stress by the product of the width of the

timber, the number of keys, and the allowable unit bearing resistance of the weaker timber on the end grain. In the calculation following, the distance the key enters either the bolster or the tension member is equal to $\frac{48,500}{8 \times 2,000 \times 2} = 1.51$ inches, or about $1\frac{1}{2}$ inches. Hence, the depth of the key is $2 \times 1\frac{1}{2}$ inches, or 3 inches. The line *g b* is cut parallel to the upper and lower edges of the compression member in order that no wedge action shall occur and increase the stress in the bolt. Since it is never desirable to have any piece come to what is known as a feather edge, or sharp angle, the point of the compression member is cut blunt, as shown at *d*. The wedge at *e*, which is used to bring the block *c* to a full bearing along the line *fg*, may be driven tight in case any shrinkage occurs in the timber of the construction.

64. Heel Joints for Trusses With Inclined Legs. In Fig. 33 are shown four detail designs that may be applied to any truss in which the lower chord is inclined. The joints in such a truss should be designed with the greatest care in order to provide rigidity during erection and thus avoid spreading and the consequent thrust on the walls. In (*a*) is given a design in which the angle between the upper and lower chords is very sharp. For this reason it is considered best to extend the upper chord to the bearing block, thus avoiding notching and the shearing stresses that would thereby exist on the tension member. Furthermore, to avoid bending moments at this point, a wrought-iron strap *a* is located centrally and parallel to the lower chord. The stress in this strap is then equal to the tension in the lower chord and must be proportioned accordingly. The requirements stated tend to reduce considerably the bearing area of the truss on the wall and to throw its center out of line with the line of the reaction, or the center of the bearing block or plate. To increase the area and make the bearing concentric with the downward force, the bolster *b* is introduced; it is secured in position by the two bolts that hold the joint together.



¾"x6" Lag Screws
a
b
(a)
a
b
c
d
e
(b)
6"x8"
15300
h
c
6"x8"
11300
a
t
1¼" Bolt
f
g
b
d
e
(c)
c'
15300
7200
b'
11300
13200
16200
a'
d'
(d)
61000 lb
b
47200 lb
a
2-1⅛" Bolts
Wr't Iron Plate
2" thick
c
2-¾" Bolts
Hardwood Key
4-⅞" Bolts
21800 lb
(e)

FIG. 33

In Fig. 33 (*b*), the lower chord is composed of two planks, one on each side of the upper chord member. In order to facilitate the designing of the connection, the strap has purposely been placed out of the center line of the tension member and is carried beyond the upper chord, which necessitates the use of a packing piece, as shown by the shape *a b c d e*. Since the upper chord rests on the wall plate, the stress due to the vertical component of the compression in this member is balanced by the reaction, and no vertical stress is created in the two members adjacent to the bearing.

In the truss shown in Fig. 33 (*c*), the members are of the same width, and the angle between them is not very sharp. The essential feature of the design is that a bolt is used to transmit the stress from the upper to the lower chord. The rafter member rests against a special casting *g*, which is made with a lip *f* to give the joint more rigidity; but it is well to neglect this lip entirely when calculating the strength of the joint. At *e* is shown another special casting that acts as a washer for the bolt and distributes the bearing on the lower chord. The lower part of this casting is stepped to avoid the necessity of making a deep cut in the timber. A strap *a* is used to hold the parts firmly together.

65. The calculations necessary to obtain the strength of the bolt *b* in Fig. 33 (*c*) are as follows: In the diagram Fig. 33 (*d*), lay off, to scale, $a'c'$ parallel to the rafter or oblique member *h*, to represent the stress in the upper chord. Draw $c'b'$ and $a'b'$ to represent the vertical reaction and the tension in the lower chord, respectively, making them parallel to the lines of action of these forces. Since the edge *i f* of the casting is made perpendicular to the direction of the upper chord, this face receives all the compression in the rafter member. The stress in the bolt may be considered as consisting of two forces, one at the end of the rafter and acting parallel with this member, as represented by $a'c'$ in the stress diagram, and the other acting perpendicular to the cut surface *d f c* of the lower chord, and represented by $c'd'$ in (*d*), which is drawn perpendicular

to the face *c d* in the detail; *a' d'* is parallel to the axis of the bolt *b*. The lengths of these lines, measured with the same scale, will therefore give the pressure on the face *c d* and the stress in the bolt *b*. The latter is found to be 16,200 pounds, which, divided by 18,000, the assumed allowable resistance of the material composing the bolt, gives .9 square inch as the required area of the bolt at the root of the thread. Since the area at the root of the thread of a 1¼-inch bolt is .89 square inch, a bolt of this size will offer the required resistance. The calculations regarding the strength of the timber required are similar to those given under previous examples.

66. In Fig. 33 (*e*) is given an excellent design for a joint. The tension of the lower chord is taken care of by the bolts *a*, which are fastened to two castings and pass outside the rafter member. The stress in the bolts is somewhat relieved by the bearing of the oblique member on the casting, as at *b*, which takes a large amount of the stress in the joint. A bolster *c* is used to increase the bearing area of the truss on the wall, and its tendency to slide along the lower chord due to the pressure on the lower part is resisted by two keys.

WALL BEARINGS AND CEILING SUPPORTS

67. Wall Bearings.—In wooden trusses, a stone bearing block or wooden plate is usually preferable to one made of cast iron, especially in cases where the truss is exposed to the elements to any degree, as in sheds, since the cast iron rusts and is apt to rot the end of the member, thereby limiting the life of the truss. In setting the stone, it is well to give the upper surface a pitch of about ⅛ inch to the foot, to prevent water from lying on top of the templet, as it might if there were an exposed eave line or a leaky roof.

68. Ceiling Supports.—To form a **ceiling support** purlins are extended from one truss to the other, usually on the lower chord. These, in turn, support furring strips, to which the ceiling is attached. Ceiling purlins are supported

by special castings and wrought-iron straps in the same manner as roof purlins. Various means are employed in securing the furring strips to the purlins, a few of which are indicated in Fig. 34. In (*a*), a cleat is nailed along each side of the purlin, on which the furring strips rest directly. In (*b*),

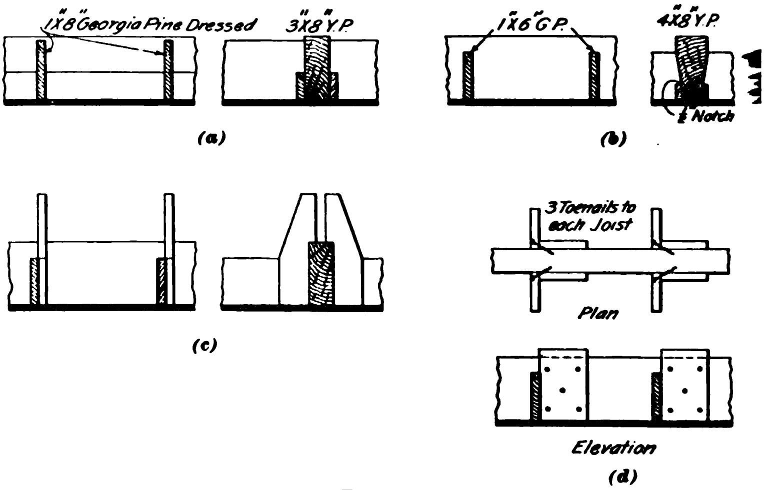


Fig. 34

the furring strips are gained into the purlin itself; this latter method is weaker and more expensive, requiring more work. In (*c*), the strips are hung on the purlins by means of wooden hooks cut to fit over the purlins. In (*d*), the strips are toe-nailed to vertical pieces nailed to the side of the purlin.

COMPOSITE-TRUSS DESIGN

MEMBERS

69. Tension and Compression Members.—In the **composite type** of construction, the upper chord is always made of timber and the tension members of wrought iron or mild steel, while the struts are usually timber, although they may be built up of structural shapes.

The method of calculating the size of the compression members is the same as that given under Timber-Truss Design. The tension members are usually composed of round or square steel bars connected either by pins to special castings or directly to the timbers. Their net area is obtained by dividing the total stress to be resisted by the safe strength of the material per square inch, a factor of safety of 3 or 4 being sufficient in this class of work. Where the net area is taken at the root of the thread, it should exceed the net area required by the calculations by about 15 per cent., to allow for the weakening of the section by cutting the thread, since cutting or scratching the surface of metal slightly reduces its tensile resistance.

70. All tension members should be provided with some means of adjusting their length so that they can be drawn tight, thus causing each rod of a system to withstand its share of the total stress. This adjustment is accomplished by the use of a thread cut on the rods, and in order to avoid weakening the member the ends may be upset, or increased in diameter, so that when the thread is cut on the end, the area at the root of the thread will exceed the size of the rod by 15 per cent. By upsetting the ends of the tension member in this way it is made of uniform strength throughout. This adds to the cost of small bars, but since the strength of any rod is calculated from the smallest area of cross-section, enough material is saved in large rods to compensate for the extra labor required.

71. There are three methods by which the length of tension members may be adjusted—by the use of nuts, clevises, and sleeve nuts or turnbuckles. *Nuts* require a square bearing, so that special castings must be employed when they are used. When the truss is pin-connected, nuts cannot be used; therefore, to tighten the tension members a rod provided with a right-hand thread at one end and a left-hand thread at the other is screwed into the clevis, as shown in Fig. 35. *Clevises* are used when one rod is desired, since their designs are such that they cannot well be used in a

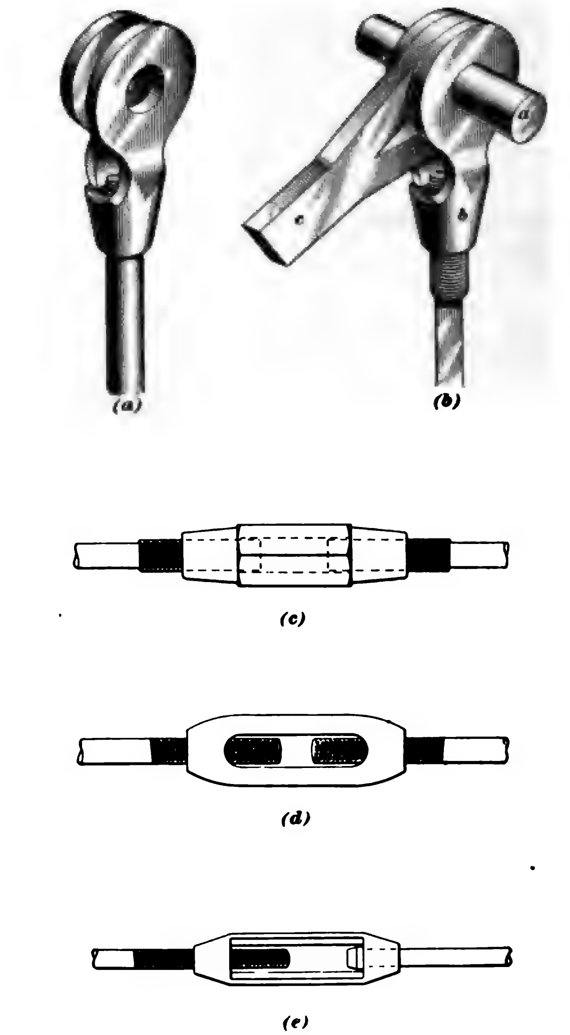


FIG. 35

position requiring two rods. In cases where two tension members are to be fastened to the same pin one of them is provided with a loop that fits between the ends of a clevis on the other member. This construction is indicated in Fig. 35 (*b*), in which *a* is the pin and *c* the loop that is placed between the ends of the clevis *b*. By this means the loads on the pin are balanced.

A *sleeve nut* is illustrated in Fig. 35 (*c*), and *turnbuckles* in (*d*) and (*e*). When either is used the tension member is made in two pieces, usually of equal size, one end of each piece being looped, and the other end upset and threaded. The threads on opposite ends of the turnbuckles should be right- and left-handed, so that when the nut is turned it will screw up on both. A disadvantage in the use of sleeve nuts is that the end of the rod is enclosed and its thread cannot be inspected to ascertain whether the rod is screwed into the nut to the full depth of the thread, while when turnbuckles are employed, the thread of the rod is exposed and its condition can readily be seen. When it is not convenient to cut a left-hand screw, a swivel may take its place, as shown in (*e*). To form the loops given in Fig. 35 (*b*), an iron bar is heated and the end bent back far enough to leave an opening the diameter of the pin, and then welded. This, of course, requires an additional length of bar, varying from $9\frac{1}{4}$ inches for a $1\frac{3}{4}$-inch rod and a $1\frac{7}{16}$-inch pin to 32 inches for a $2\frac{1}{2}$-inch rod and a $5\frac{7}{8}$-inch pin.

CONNECTIONS

72. Joints in General.—Since it is practically impossible to form an absolutely rigid connection between small metal tension rods and wooden compression members in composite trusses, use may be made of *adjustable joints*, or joints in which the members are free to move to a limited extent. In such cases, although the members can adjust themselves to the strain, a great deal of the rigidity of the joint is sacrificed, necessitating the use of strong cross-bracing between trusses.

73. Heel Joints.—In Fig. 36 are given the details of a number of **heel joints** that may be used in composite trusses. In (*a*), the lower chord is raised, as in a church truss, the compression member being inserted between the two lips shown, and held in place by lagscrews. These screws simply hold the casting in place during erection, and are not subject to any strain after the truss is in place. In (*b*) is given a detail of a shoe similar to the one in (*a*),

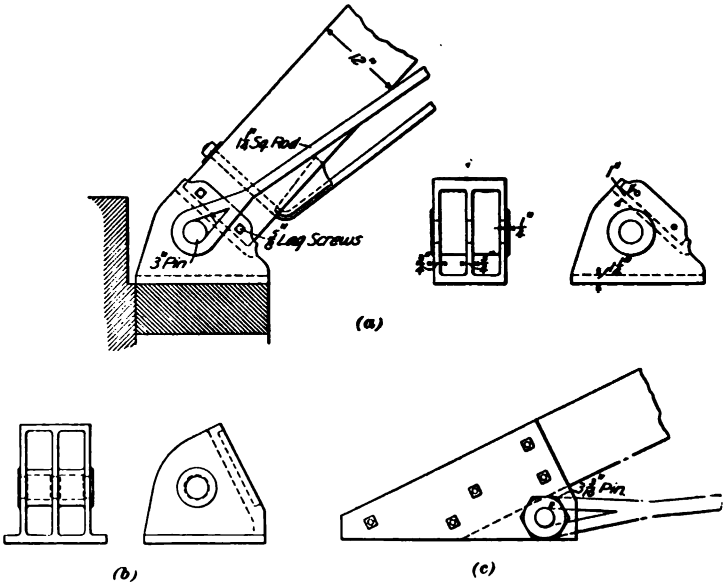


FIG. 36

while in (*c*) steel plates are connected to the wood by bolts, whose strength for both bending and shear should be calculated.

74. Strut Joints.—Many of the **strut joints** given for timber trusses may be employed in composite trusses. For example, methods given in Figs. 11, 12, 13, and 14 may be employed for the upper ends of struts of composite trusses of the Howe form. The joints illustrated in Fig. 37 (*a*), (*b*), (*c*), (*d*), and (*e*) are used to secure struts and ties in Fink trusses. In (*a*), the strut *b* rests against a bolster *d* that

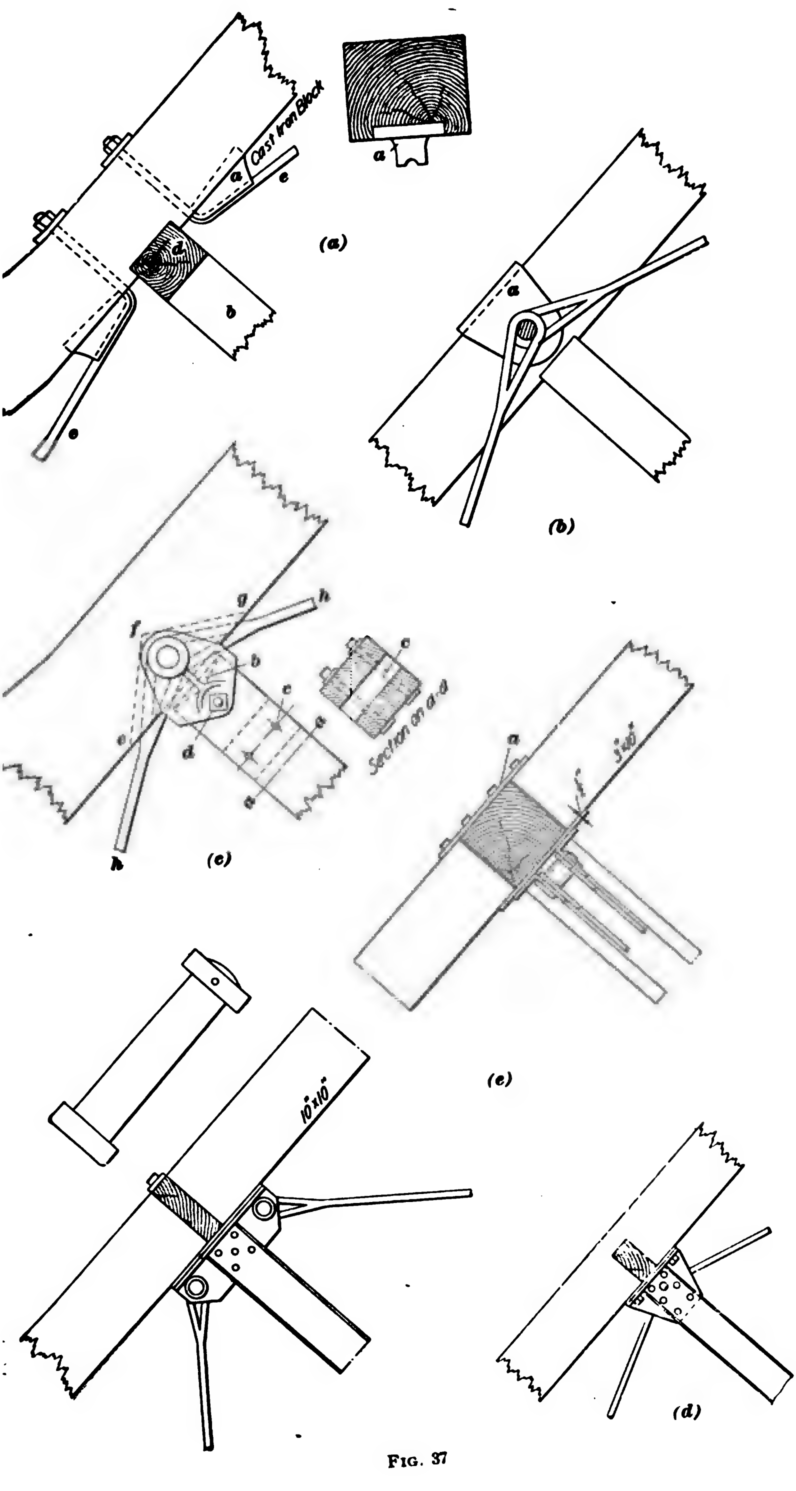


Fig. 37

may be avoided by using a casting in its]

In Fig. 37 (*b*), the strut is framed d upper chord. To strengthen the joint, a is slipped over the compression member at the panel point, and to increase the be casting were omitted, the diameter of determined by the compression on the chord. By the use of the casting, th sufficient bearing for the pin is of secc the size of the pin being determined t shear or bending. Its diameter may siderably smaller than when no casting is

In Fig. 37 (*c*) is given a pin connection in which the casting used serves not onl the strut but also holds the pin to which bers are fastened. In this connection th times the rafter members are made up timber separated from 2 to 4 inches, by whole being held together by bolts. W the section shown, a lip *b* just the width of the ends of the two timbers composing provided in the casting. This projection two members apart, though a wooden se placed adjacent to the casting, as shown holds the casting in place on the end of facilitates erection, besides preventing

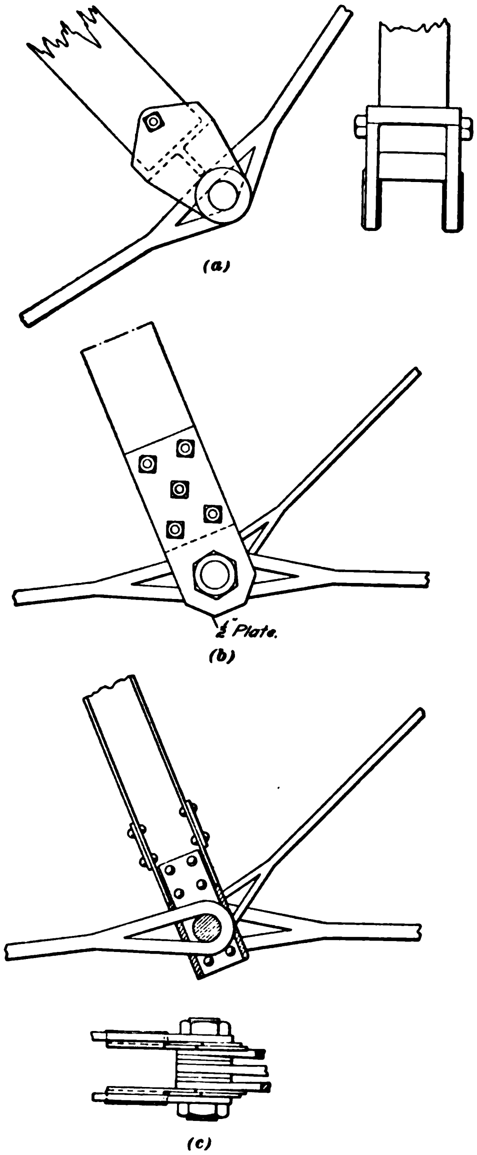


FIG. 38

75. When the strut is of structural steel, such a joint as the one in Fig. 37 (*d*) may be used to advantage, the upper end of the strut serving to hold the pin. In the detail (*e*) two pins are used and the members retain positions corresponding to those in the frame diagram, the necessity of cutting the compression member being avoided without causing the design to loose all the advantages of a pin connection. After the pin is in place a sleeve is slipped on, and secured in place by a taper key driven into a small hole bored through both pin and sleeve. If desired, a split pin may be used, since the purpose of the sleeve is simply to hold the pin firmly to prevent displacement. When two rods are used, they are kept apart by a separator, with which the pins should be provided. The ends of the purlins are strapped to the main rafter member and to each other by means of a piece of iron bar *a*. These purlins are supported by the flat plate that caps the strut.

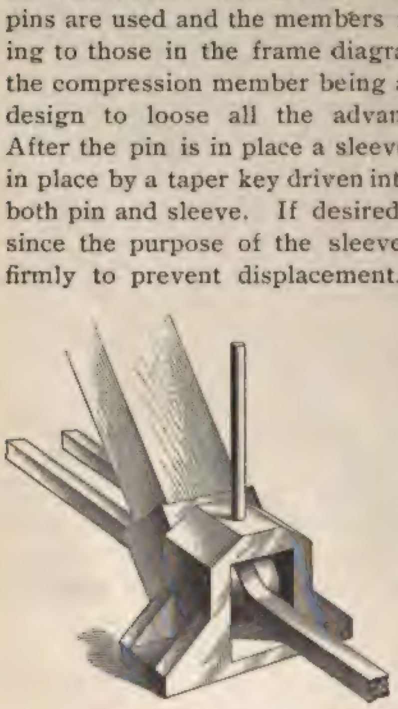
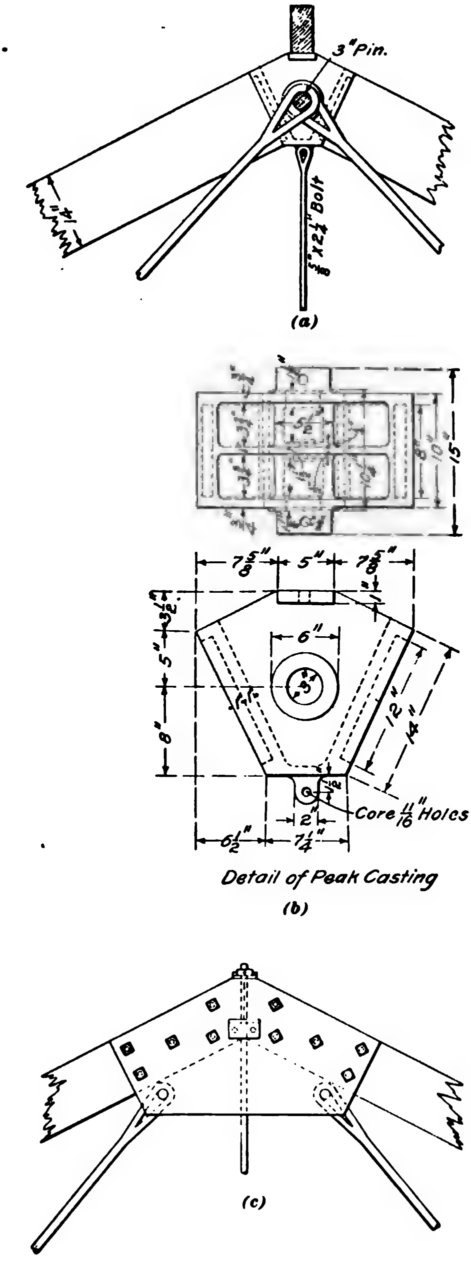


Fig. 39

76. In Fig. 38 are given three designs for the lower end of the strut of a Fink truss. The casting in (*a*) is similar to the one in Fig. 37 (*c*), while in (*b*) plates bolted to the sides of the strut are used, and in (*c*) the strut is built up of two channels having a pin passed through the end. As no new features are here introduced, further explanation is unnecessary.

77. A convenient method of framing the members of a Howe truss is shown in Fig. 39. A cast-iron box, provided with holes for the pin and brackets to receive the ceiling purlins, is used. The strut is inserted in a bearing bracket, and the vertical tension rod passes through the box and is held in place by a nut on the inside of the box. The



3" Pin.
5/8" x 2 1/4" Bolt
14"
(a)
15"
10"
8"
7 5/8"
5"
7 5/8"
1"
3 1/2"
5"
8"
6"
12"
14"
1/2
2"
Core 11/16" Holes
6 1/2"
7 1/4"
Detail of Peak Casting
(b)
(c)

Fig. 40

tension members are put in position and the pin driven through the casting. The angle of the bearing bracket is altered for the particular panel in which it is to be placed.

78. Peak Joints.—Since compression members cannot be continuous at the peak, some means of holding the pin in place must be adopted in pin-connected trusses. In Fig. 40 is given a detail in which a casting is employed to hold the ends of the compression members and the pin. From (*b*) it can be seen that two lips are formed on the top of the casting to support the ridge pole. In (*c*) the pins are held by two wrought-iron plates bolted to the sides of the compression members. The designs in Fig. 18 are applicable to composite trusses as well as to those composed entirely of wood.

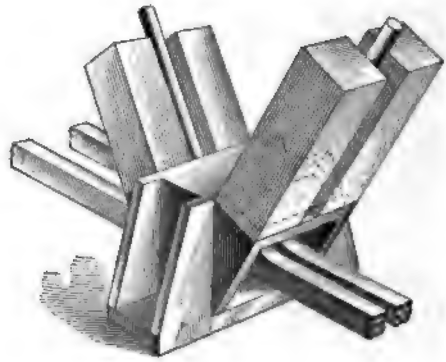


FIG. 41

79. Center Joints.—Some form of casting is usually employed at the **center joints** of Howe trusses. The design shown in Fig. 41 corresponds with the design for the strut joint illustrated in Fig. 39.

STEEL-TRUSS DESIGN

TWO METHODS OF CONSTRUCTION

80. All trusses whose members are composed of steel or wrought iron belong under this subdivision. Formerly wrought iron was extensively manufactured, since nearly all the structural shapes were made of it, but on account of the superior strength of steel and the cheapness of its manufacture since the introduction of the Bessemer process, as

well as the convenient shapes obtainable, it readily adapts itself to truss construction, and hence has now almost superseded wrought iron in the market.

81. Steel trusses may be constructed with either riveted or pinned joints, the former being preferable for short spans, because the several members may be riveted together in the shop and the truss shipped in sections. Such frames are put together with ease and rapidity, and since the field work is greatly lessened and the shop work is cheaper than that done on the field, the total cost is decreased. They have the additional advantage of being stiffer laterally than pin-connected trusses, since the joints do not give. In trusses of longer spans, the members are necessarily of such great length that if two were joined together shipment would be almost impossible, and the work in the field would be greatly increased. In such cases it is well to use pin-connected joints, fitting the parts so carefully in the shop that they can be readily assembled in the field.

It is customary to add $\frac{1}{32}$ inch of metal to all surfaces in trusses exposed to the action of corroding gases. Pin-connected trusses are preferable in many instances since their tension members are usually round or rectangular in section, thus exposing less surface to corrosion than do angles figured to withstand the same tension.

MEMBERS

82. Riveted Tension Members.—The shapes most commonly employed for **riveted tension members** in steel-truss connections are angles, flat bars, and, occasionally, channels. Angles, being stiffer than flat bars and having less section than channels, are used to the greatest extent, usually being placed in pairs, back to back, as shown in Fig. 42 (*a*), which is a section through *a a* in (*b*).

To provide greater rigidity and prevent the angles from striking against each other during vibration, small washers, called *separators*, are placed at intervals of $2\frac{1}{2}$ or 3 feet

between the angles, as at *b* in (*b*). These consist of small pieces of plate, round or square, having holes of the same diameter as the rivet holes punched through the center, and should be of the same thickness as the gusset plates to which the tension members are fastened, and held in place by rivets driven through both angles and separators.

83. The building laws of some cities provide that when an angle is connected by but one leg, the strength of that

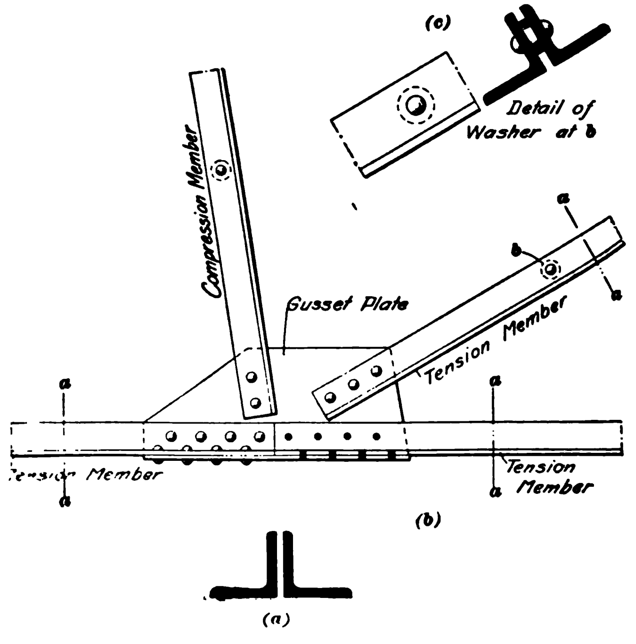


FIG. 42

leg only is to be considered when figuring the resistance of the angle, it being claimed by some that as the angle is eccentrically loaded, it is liable to tear through the rivet holes, as would a piece of perforated paper. At any rate, it is always preferable to connect both legs of the angles by splice plates, cover-plates, or splice angles, which connections will be fully explained later.

84. In riveted trusses, flat bars are seldom adopted for the lower chord, but they may be used for tie-members in cases where there is no liability of the stress being changed to compression by the eccentric loading of the truss from the action of the wind or other causes. In calculating the strength of flat bars as tension members, the whole section may be considered and the net section obtained by deducting for rivet holes. The only advantage gained by using these flat bars is that less riveting is required than when the members are composed of angles placed back to back, as there is no projecting leg to connect by means of plates or splice angles. When the lower chord supports a floor load distributed over its length, it is well to use channels placed back to back, and in calculating the size of the member, to consider its ability to resist bending as well as tensile stresses.

85. Pin-Connected Tension Members.—When there is no danger that the stress in the tension member of a pin-connected truss will change to compression, round, square, or flat bars of either iron or steel may be used; but if the stress is liable to change, channels used in pairs are preferable. The round and square bars are employed as tension members only when the stresses are comparatively light, their use and the means of adjustment having been fully treated under Composite-Truss Design.

When the span of a truss is long and the stresses to be resisted are great, pin connections are commonly adopted, and a suitable tension member is formed by a flat bar of rectangular section, such as is illustrated in Fig. 43. For the

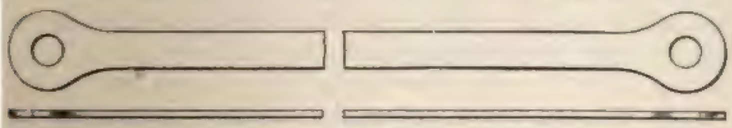


FIG. 43

sake of economy it is well to make the panels of equal length, and by varying the number of bars in the panel, it is often possible to use bars of the same size for several different members. All machine work on these tension bars,

must be done with the greatest
. . . specify that the distance
. . . les in the several bars shall be
. . . of the metal is 64 . . . A more
. . . vever, is that the pin holes fall
. . . the other, when the bars are
. . . a pin having a diameter . . . inch
. . . will pass through them all with . . .
. . . of proper length and pinholes of
. . . tance of which can readily be seen
. . . there is no way of adjusting the

. . . and obtain accuracy in the shop. It
. . . number of these bars together and
. . . at the same time. The enlargement
. . . should be obtained by upsetting, and
. . . throughout the member. The
. . . should be such that when the hole
. . . on a transverse line through the
. . . per cent. more than that of the
. . . have . . . inch clearance and must
. . . to the plane of the rod. When
. . . which several bars are to be placed
. . . adopted should be the one that
. . . on the pin, and a space of
. . . exist between tension bars in
. . .

. . . tension members should lie in
. . . of the truss, but when this is
. . . from the plane of the truss should
. . . the foot.

. . . tension members are usually lat-
. . . enough to extend beyond the
. . . cutting . . . the flange at the
. . . when . . . practice becomes
. . . be . . . with pin plates
. . . strength of the member will not be

diminished. These pin plates also serve to reduce the unit pressure on the pin to the allowable limit, or even lower; they should be long enough to properly distribute the stress to the rivets connecting the plates to the channel, and should extend no less than 6 inches within the tie-member so as to provide for at least two transverse rows of rivets. The net section at right angles to the axis of the member and through any pinhole in these riveted tension members,

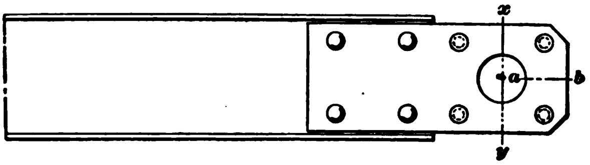


Fig. 44

as along the line xy, Fig. 44, should be at least 40 per cent. greater in area than the net section of the body of the member, and the net section along the axis ab should be at least 70 per cent. of the net section along xy. Members of this character should not be spliced unless absolutely necessary, in which case the splice must be proportioned to the full strength of the member instead of to the stress to be resisted.

87. Riveted Compression Members.—The sections usually employed for compression members in riveted trusses are given in Fig. 45 (*a*), (*b*), (*c*), and (*d*). In (*a*), two angles are placed back to back and held apart by separators *a*. These separators are of greater importance in compression than in tension members, as they cause the angles to act in unison, increasing the radius of gyration of the section and consequently the strength per square inch. This can be clearly seen by referring to any table giving the properties of angles. Were the angles shown in (*a*) to act separately, their least radius of gyration would be about the axis xx and would be equal to .86, while when separators are used the least radius of gyration of the section is about the axis yy, and is 1.71, an increase of nearly 100 per cent. In compression members, separators are usually placed along

the whole length of the member at any distance not exceeding eight times the shortest leg of the angle used. The section in (*a*) is most commonly used, as it can readily be put together and because it affords a section of small area for roof trusses whose stresses are light.

When the loads are heavy, or a load must be placed between the panel points, the section in (*a*) may be strengthened by the introduction of a web-plate between the angles, as in (*b*), or channels placed back to back, as in (*c*), may be substituted. When the form in (*b*) is used,

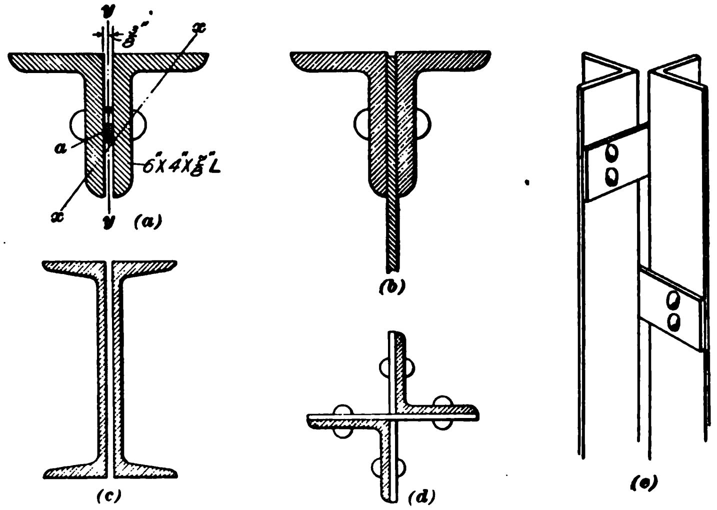


FIG. 45

rivets should be placed along its length at intervals of not more than sixteen times the thickness of the angle, and in no instance should the pitch be more than 6 inches.

88. The section usually employed for struts is the same as that for compression members shown in Fig. 45 (*a*), although that indicated in (*d*) is used to some extent. In this case the angles are placed diagonally opposite each other and are fastened together by means of plates, two rivets to an angle in each plate, as shown in the elevation in (*e*). When the upper chord must be heavy or is subjected to cross-bending

ress, channels placed back to back are suitable, and they ay also be used to advantage for long struts in riveted ork. When the load carried by the strut is light, one angle sometimes used, although ordinarily this is poor practice, s the stress is imposed on one side of the truss. When a ngle angle is used as a compression member, the legs of e angle should be equal, in order to have the greatest posble radius of gyration with the use of the least material.

In splicing a compression member composed of angles laced back to back, it is advisable to connect both legs, as tension members. In no case should the radius of gyraon of the section selected be less than $\frac{1}{150}$ of the length.

When calculating the strength of compression members e full area should be considered, no deduction being made r rivet holes, since shop rivets are commonly employed long the length between panel points, and these, when proprly heated, completely fill the holes and bear their share of e compressive stress.

89. Pin-Connected Compression Members.—These embers are usually made up of channels placed back to ack for the chords, and with the backs on the outside for the truts. By this arrangement the struts can be conveniently ntered between the backs of the chord members, thus avoidg the necessity of cutting away the flanges at these joints.

Sometimes the upper chord has a cover-plate on top and is tticed at the bottom. This, however, is unnecessary in ost roof trusses unless the span is so great that it is desirale to increase the area of the section in this way. The istance between channels should be such that the radii of yration about the two axes are equal, although the great dvantage gained by keeping the truss of uniform width roughout should not be sacrificed for this purpose. ver possible, the width of the truss should be sucl adii of gyration of the largest compression me qual about both axes.

Although channels are used almost en ong span, in the shorter spans, where th

required to withstand great stress, the struts are composed of two angles placed back to back.

90. As in tension members, pin plates are used whenever a compression member connects with a pin, as shown at *a*, *a* in Fig. 46. The pin plates serve the double purpose of reenforcing the section of the pin and reducing the unit bearing stress on the metal adjacent to the pinhole. Where the load is light, one pin plate is used, but for heavier loads it is preferable to place one on each side of the web of the channel. When it is necessary to place a tension member next to the compression member, countersunk rivets may be used, and in such cases, the plate in which the rivets are countersunk should be at least $\frac{7}{16}$ inch thick. The channels and pin plates should extend far enough beyond the pin to permit at least two rivets to be driven to hold the pin plates and web together. In figuring the strength of these compression members, the usual column formulas should be employed. When it is found necessary to splice a compression member, the splice is made equal to the full strength of the member, and no reliance is placed on the ends of the channels abutting.

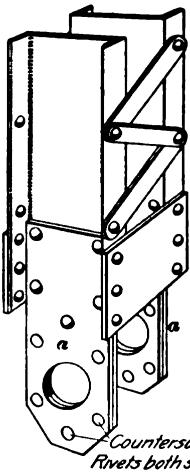


FIG. 46

91. Splices in Riveted Trusses.—The simplest form of splice used in joining tension and compression members consists of a plate riveted between the legs of the angles. No figure is given to illustrate this as it is the same as that given in Fig. 47 (*a*), except that the cover-plate is not used. The strength of this splice is limited, however, as it connects but one leg of each angle, and hence, where greater stress is to be resisted the form shown in Fig. 47 (*a*) may be employed with advantage, since by its use the load may be transferred concentrically to the next member.

By using splice angles, as in (*b*), there can be secured a

strong and efficient splice, which may be still further strengthened by riveting on a cover-plate. If the angles are not of the same thickness the thinner must be built out to the

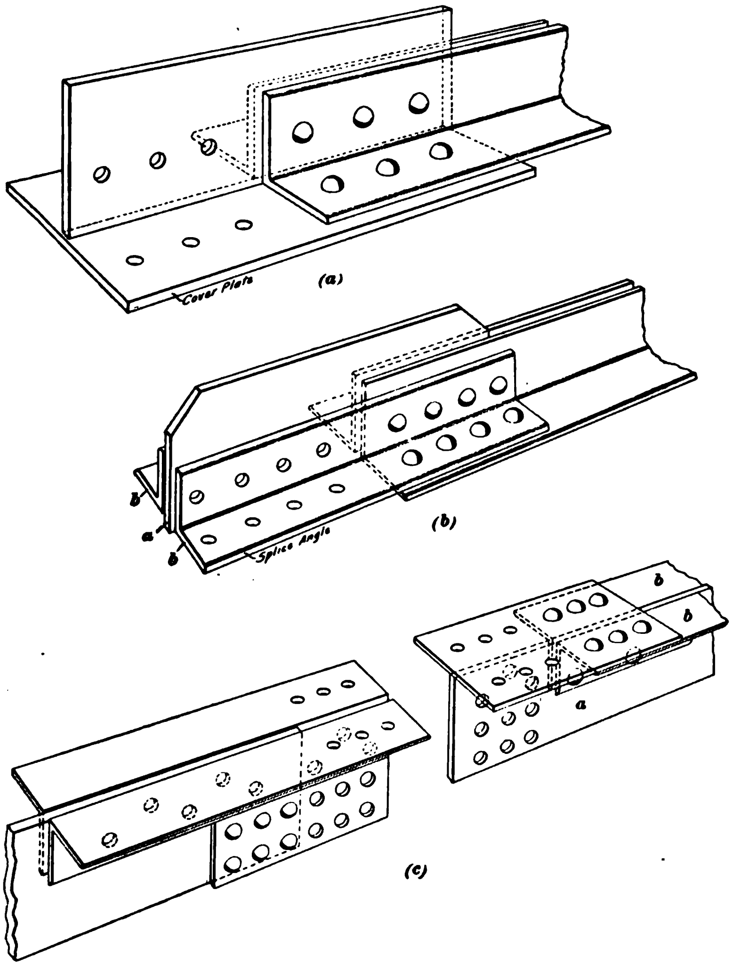


Fig. 47

size of the larger by using packing plates, although if they do not vary more than $\frac{1}{8}$ inch the splice angle may be swaged or bent to take up this difference.

When the section illustrated in Fig. 45 (*b*) is adopted, it is well to use the splice shown in Fig. 47 (*c*). This is made by carrying the web *a* beyond the ends of the angles *b*, *b*, thus forming a rigid connection. In order to avoid reduction of area at the splice, it is necessary to introduce splice plates on either side of the web, and sometimes a cover-plate also, as indicated in the figure.

The plates and angles used in making a splice must take care of the entire stress, the strength of the members themselves being entirely disregarded, and the strength of the splice must be proportioned to the full strength of the tension or compression member, and not simply to the stress that they may be called on to resist. This makes the strength of the member uniform throughout its length.

92. The strength of a splice may be figured as follows: When a tension member is being considered, the strength of the section should be computed, and if a splice plate is to be used the number of rivets required is determined by dividing the strength of the tension member by the bearing value of one rivet, or by the shearing value of one rivet in double shear if the tension member is made up of two pieces; these two values should be ascertained and the smaller one used in determining the number of rivets. The length of the splice plates or angles is readily obtained when the number of rivets and spacing are known. The splice plate should be of such thickness that its strength in web bearing is not exceeded by the strength of the rivet either in double shear or in bearing value on the rolled shapes that compose the tension member. The net area of the required splice must be made equal to the net area of the tension member. The rivets should be so distributed that the approximate center of gravity of all, taken collectively, will be near the center of gravity of the rolled shapes in the tension member.

93. The method of designing the splice for compression members in a roof truss is similar to the above with the exception that the strength of the member is figured for the full area of the angles, the splice plates being made equal in

area. The following example clearly illustrates the principles involved:

EXAMPLE.—What is the tensile strength of the connection shown in Fig. 48?

SOLUTION.—Using ¾-in. rivets, the net allowable area of the angle would be $3\frac{1}{2} \times \frac{3}{8} - \frac{7}{8} \times \frac{3}{8} = .984$. Assuming 14,000 lb. per sq. in. as the tensile strength of the material composing the member, the strength of each angle is $.984 \times 14{,}000 = 13{,}776$ lb., and of two angles is $2 \times 13{,}776 = 27{,}552$ lb. If the unit shearing value of the rivets is 10,000 lb., the strength of one rivet in double shear is equal to twice

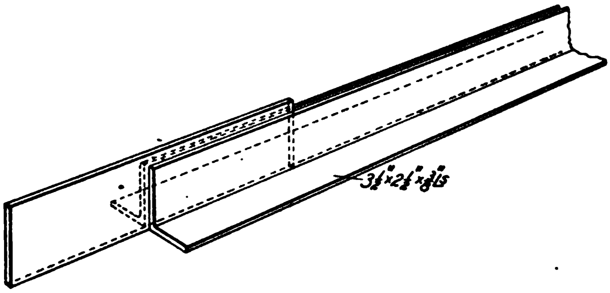


FIG. 48

the product obtained by multiplying its area in cross-section by 10,000. The area of a ¾-in. rivet is .4418 and its strength in double shear is, therefore, $.4418 \times 10{,}000 \times 2 = 8{,}836$ lb. The number of rivets required is $\frac{27552}{8836} = 3$ rivets. The required thickness of the plate, allowing 20,000 lb. per sq. in. for the bearing value, is $\frac{8{,}836}{20{,}000 \times \frac{3}{4}} = \frac{9}{16}$-in. plate. Since the bearing of the rivets in the angles is ⅜ in., it is evident that they are sufficiently strong. The strength of the connection is therefore 27,552 lb. Ans.

94. Riveted Strut Joints.—The number of structural shapes available for steel trusses limits the variety of forms for strut joints, so that those shown in the figures accompanying the text fully illustrate the principles involved in their design. The general remarks that were given concerning wood and composite trusses apply also to the design of structural strut connections.

The designer should endeavor to so place the angles or flats composing the truss that their center of gravity corresponds with that of the frame diagram. The number of

rivets required to resist the stress in the member is then calculated, the strength, both in shearing and bearing, being analyzed to determine which is lower. In cases where the chord member continues through two panels, the number of rivets required is determined by the difference in stress between the two panel lengths.

95. The ideal position for rivets is along the neutral axis of the angle. However, this is impossible in actual work, because the center of gravity of an angle section is so near the back that there is not room enough to form the rivet head. It is not customary, therefore, to place the rivet hole less than $1\frac{3}{8}$ inches from the back of the angle when a $\frac{3}{4}$-inch rivet is used.

After determining the number of rivets required in each member meeting at a joint, the size and shape of the plate depends on the spacing adopted. The distance between centers should never be less than three times the diameter of the rivets, but never more than 6 inches. The shape of the plate may be altered by varying the spacing of the rivets, which must frequently be done for economy, or for the sake of a better appearance. It is often found that if the shape of the plate is made to indicate the shape of the panel of the truss in which it is placed, the general effect will be more pleasing.

The simplest form of a strut joint in a structural steel truss is shown in Fig. 49. The dot-and-dash lines indicate the neutral axes of the angles, and as can readily be seen, they intersect at the panel point. The edges $g\,h$, $f\,j$, and $f\,g$ have been cut parallel to $b\,a$, $c\,a$, and $d\,e$, respectively, which gives the plate a finished appearance and adds character to the design.

96. The following method for determining the number of rivets in the strut connection, Fig. 49, is the one commonly employed for such work. The size of the rivets and the amount of stress they withstand must first be determined. Since roof trusses are not so liable to be subjected to sudden shocks or loads suddenly applied as are trusses

used in railroad bridges, a large allowable unit stress may be assumed.

It will be noticed that there are some field rivets, or rivets driven while the work is being erected, used in this connection. In conservative practice field rivets that are included in the design are assumed to have five-sixths of the allowable shearing value of shop-driven rivets. The reasons for this are that the work cannot be done as carefully on the field as in the shop, the holes frequently are not in alinement, and the heated rivets must often be carried so far that

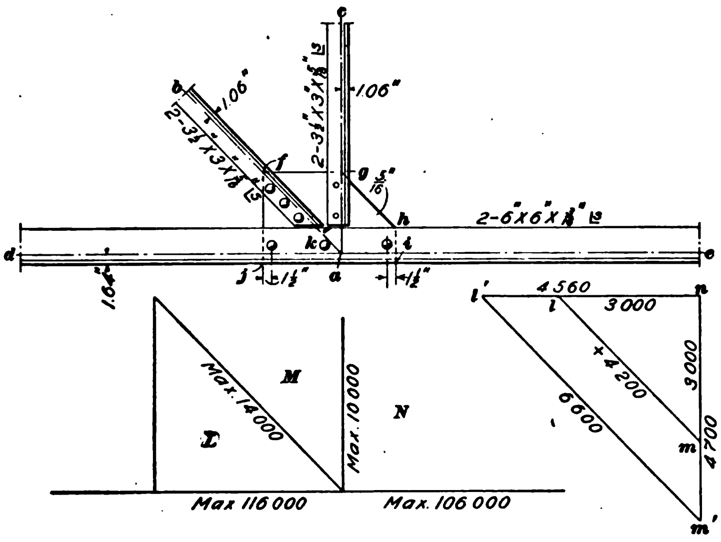


FIG. 49

they are cooled too much to be driven successfully. Other reasons for the inferiority of field rivets are that the machine used in driving them is very light, and that in some cases they must even be driven by hand. When considering the bearing value of the rivets on the plates or rolled shapes, no distinction need be made between shop and field rivets, from the fact that the defects due to field riveting are in the rivets themselves, and the field rivets have as good bearing as those driven in the shop.

The allowable strength of $\frac{3}{4}$-inch rivets, based on material having an allowable unit tensile value of 15,000 pounds, is given in Table II.

TABLE II

Rivet				Plate		
Diameter Inch	Kind	Shear		Thickness Inch	Bearing	
		Single Pounds	Double Pounds		Ordinary Pounds	Web Pounds
$\frac{3}{4}$	Shop	4,788	9,577	$\frac{5}{16}$	4,570	6,094
	Field	3,990	7,980			

The maximum stress in the member *c a* is 10,000 pounds, but though the rivets used are field-driven and in double shear, the bearing value of 6,094 pounds must be taken, as it is less than the double shearing value of a field rivet. From this it is evident that two rivets are sufficient for the connection, while the member *b a*, having a stress of 14,000 pounds, will require three.

The number of rivets used for the lower chord should be sufficient to provide for the difference in stress between the portion of the chord at the right of the connection and the portion at the left, which stress is equal to the algebraic sum of the horizontal components meeting at the connection. The difference in stress between the two portions of the tie-member is, in this instance, 116,000 − 106,000 = 10,000 pounds, so that practically two rivets are required. But since the maximum pitch of rivets is 6 inches and the two rivets are placed so near the ends of the plate that there is a space greater than 6 inches between them, another rivet should be used at the center, as shown at *k*.

97. In cases where the lower chord is not continuous, as in Fig. 50, it is preferable to connect both legs of the angle, not only to give greater strength to the splice, but

also to provide greater lateral stiffness for the truss. The stresses in the several members being given in the figure, the number of rivets required may be calculated as in the following example:

EXAMPLE.—What number of rivets will be required in the connection shown in Fig. 50, if $\frac{3}{4}$-inch rivets are used throughout the joint?

SOLUTION.—The values for rivets and plates given in Table II are used in this case also, but the number of rivets in the connection of the member *do* must be determined in a different manner from that just shown. Here it is convenient to place three rivets through the vertical legs of the angles; therefore, the strength of these three rivets must be determined and also the remaining stress to be provided for by the rivets in the flanges or horizontal legs of the angles. The strength of the three rivets in the vertical leg of the angle is equal to 3 × 6,094 lb. = 18,282 lb., so that if the total stress to be resisted is

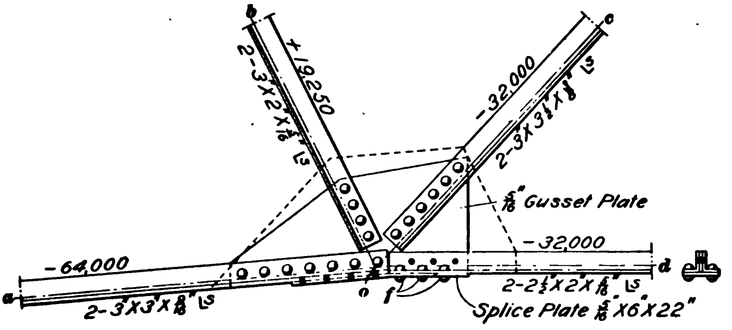


FIG. 50

32,000 lb., the balance to be sustained by the rivets in the flanges is equal to 32,000 − 18,282 = 13,718 lb. These rivets are in single shear and ordinary bearing. The single shearing values for $\frac{3}{4}$-inch shop- and field-driven rivets are 4,788 and 3,990, respectively, while the bearing value is 4,570. Since the lowest of these values must be considered in the analysis, and as the rivets *f* under discussion are shop-driven rivets, their strength will be taken at 4,570 lb. Then, as the total stress that these rivets must resist is 13,718 lb., three rivets are required, but to make the connection symmetrical four rivets must be used, two on each side. Since the member *do* is connected to *ao* by the $\frac{5}{16}$-inch splice plate, it would be well, in order to realize the full strength of the splice plate, to introduce two additional $\frac{3}{4}$-inch rivets in the lower flange of the angle, so that six rivets, three on a side, are used in this connection. Ans.

Since the rivets are in double shear, and the gusset plate is only

$\frac{5}{16}$ inch thick, the shearing strength of the rivets will not be realized, and their bearing value on the plate must be used in the calculations. This is evident, for the allowable web-bearing value of a $\frac{3}{4}$-inch rivet in a $\frac{5}{16}$-inch plate is 6,094 lb. This value will govern the number of rivets, because the smallest angle in the connection capable of bearing 4,570 lb. is $\frac{5}{16}$ inch in thickness, and as there are two angles, one on each side, the bearing value for one rivet in ordinary bearing will be 9,140 lb. When the double shearing value of the rivets alone is considered, the number of rivets required for the member *bo* is 19,250 ÷ 6,094, or 4, and the number required for the member *co* is equal to 34,000 ÷ 6,094, or 6.

Similar calculations may be made for the number of rivets in the member *o a*, with the result that this connection is designed as shown.

The joint whose analysis is given above is used in a Fink roof truss, the shape of the plate being determined by the

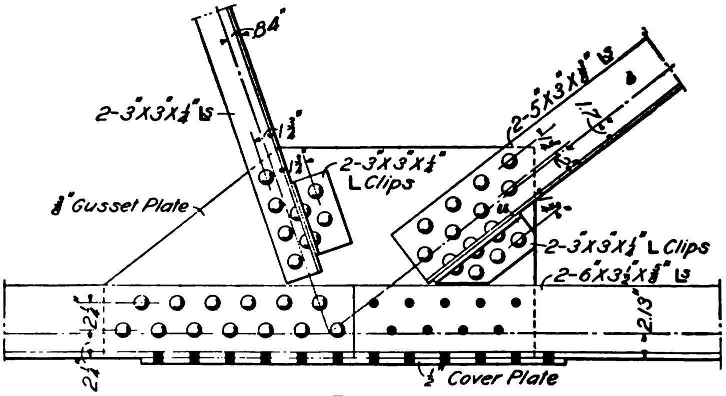


FIG. 51

number and spacing of the rivets, the rivets in this case being placed as close together as possible. By lengthening the gusset along the center line of the member *o b* and making the cuts parallel to the members of the truss, a design can be obtained that might be considered superior to that shown in Fig. 50. This change of design is indicated by the dotted lines, although the effect could more readily be noted if the whole frame diagram were given.

98. When the stresses are high the angles may be connected to the gusset plate by means of angle clips, as in Fig. 51. It is of advantage to make such a clip and the

angle it connects of the same size, and to use the same number of rivets in each, or better still, to so arrange the rivets in both angle and clip that their moments about the center of the connections are equal. By this method the rivets share the strain equally, for the neutral axis of the angle is concentric with the line of resistance of the rivets. This point is demonstrated in the following example:

EXAMPLE.—In Fig. 51, is the load on the member *b* concentric?

SOLUTION.—The distance of the center of gravity from the back of the 5″ × 3″ × ⅜″ angle is 1.70 in., and the distance from the three rivets shown in the angle clip to the back is 1.75 in. The moment of these rivets about the axis of the member may then be taken in units of rivets multiplied by the distance from their center line to the center of gravity of the angles forming the member. The moment of these three rivets in the clips will equal $(1.7 + 1.75) \times 3 = 10.45$. The distance from the first row of rivets in the 5″ × 3″ angles to the back of the angles is 2 in., and the distance from the second row to the same place is 3.75. The combined moments of these two rows of rivets is $4 \times (2 - 1.7) + 4 \times (3.75 - 1.7) = 9.40$. This shows that the moments of the several rows of rivets about the axis of the member nearly balance each other and thus insure a concentric load on the member. Ans.

It will also be noticed that if it were necessary to place another rivet in the angle clip, as at *a*, three rivets would have been driven in order to realize the full strength of this rivet, because for each rivet driven in the outstanding leg of the angle clip, one rivet should be driven in the vertical leg of both the angle clip and the member.

99. Purlin Connections.—Fig. 52 shows the usual method of connecting the struts and ties when the upper chord is made of two angles and a web-plate, and also gives an excellent method of supporting the roof purlins. The purlins on such a roof are from 4 to 6 inches in width, so that by placing the angle brace *a* 2 or 3 inches below the panel point, the load is brought centrally over the joint, as it should be. This brace is composed of two vertical angles riveted between two horizontal angles, or bent plates cut to shape, the latter in turn being riveted to the upper chord. Provision is made for bolting the purlin to the clip by leaving open holes in the vertical angles.

100. Purlins are usually made of angles, **Z** bars, or I beams. *Angle purlins* are used with advantage when loads are light, and the connections are made so that the center line of the load falls in line with the panel point, as in Fig. 53 (*a*), an angle clip being used to hold the angle securely to the frame. To give greater stiffness to the design angle purlins should always be placed with the flange pointing up the slope.

The **Z** bar is best adapted for use as a purlin, since its shape gives great strength. It should not be placed as shown by the dotted lines at *m* in Fig. 53 (*b*), since less strength would be developed and a groove would be formed between the web and the lower flange, in which water of

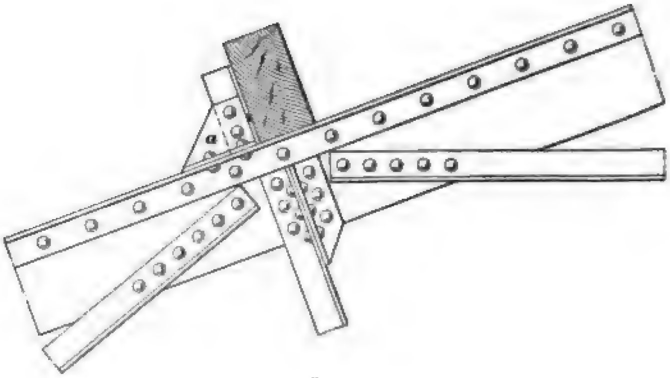


FIG. 52

condensation could collect if there were any leakage whatever from the roof. The connection to the frame is made by riveting through the lower flange, and if two **Z** bars abut at a truss, further connection is afforded by riveting a splice plate to both webs of the **Z** bars, and greater strength may be gained by riveting the upper chord, as shown dotted in the figure, although this is not absolutely necessary.

I beams may also be used as purlins, but are not as efficient as **Z** bars. Connections may be made by riveting through the lower flange to the upper chord of the truss, and they are further secured by a splice plate connecting

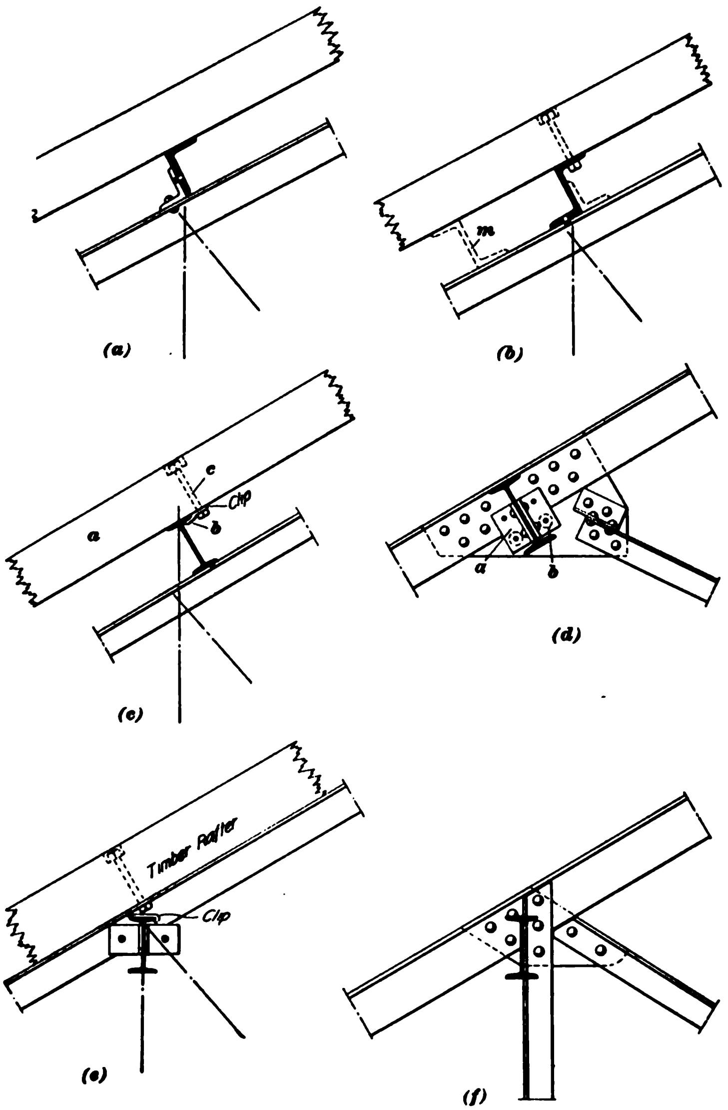


FIG. 53

two abutting purlins. A detail of an I beam employed as a purlin is illustrated in Fig. 53 (*c*); here it can be noticed that the secondary wooden rafters *a* are held to the purlins by means of a bar-iron clip *b* and a bolt or screw *c*. The I beam may be riveted to the upper chord so that the upper flange is on a plane with the upper chord by using two angle clips, as in (*d*). In this case, the clips are usually riveted to the I beams in the shop, and open rivet holes are left in the upper chord for field connections. In (*e*) and (*f*) are given two methods for connecting I beams when the purlin is placed in a vertical position.

CONNECTIONS

101. Heel Joints.—It is as important in steel trusses as in those composed of wood that the center lines of the various members intersect, and that the wall bearing be directly under the panel point and symmetrically placed with respect to that point. The **heel joint** is even of greater importance in steel construction than in wood, especially in trusses of large span, since the stresses to be provided for are much greater than in any type described. The joint at the heel must be figured not only for the tension and compression in the chords meeting at this point, but also for the shear and bending due to the reactions. In large trusses the shear is great, and considerable ingenuity is sometimes required to provide for it successfully.

102. A simple form of heel joint could be made by carrying the rafter member down to the wall plate, but since in steel-truss construction this upper chord is made of angles, the bearing would be too small and some means must be adopted to provide a bearing that would have sufficient area and still be concentric with the reaction.

In Fig. 54 (*a*) is given an illustration of a heel joint for a truss resting directly on a wall. The gusset plate has been carried above the top line of the upper chord, and accommodates an angle clip that distributes the load of the chord throughout the plate, and makes the resistance concentric

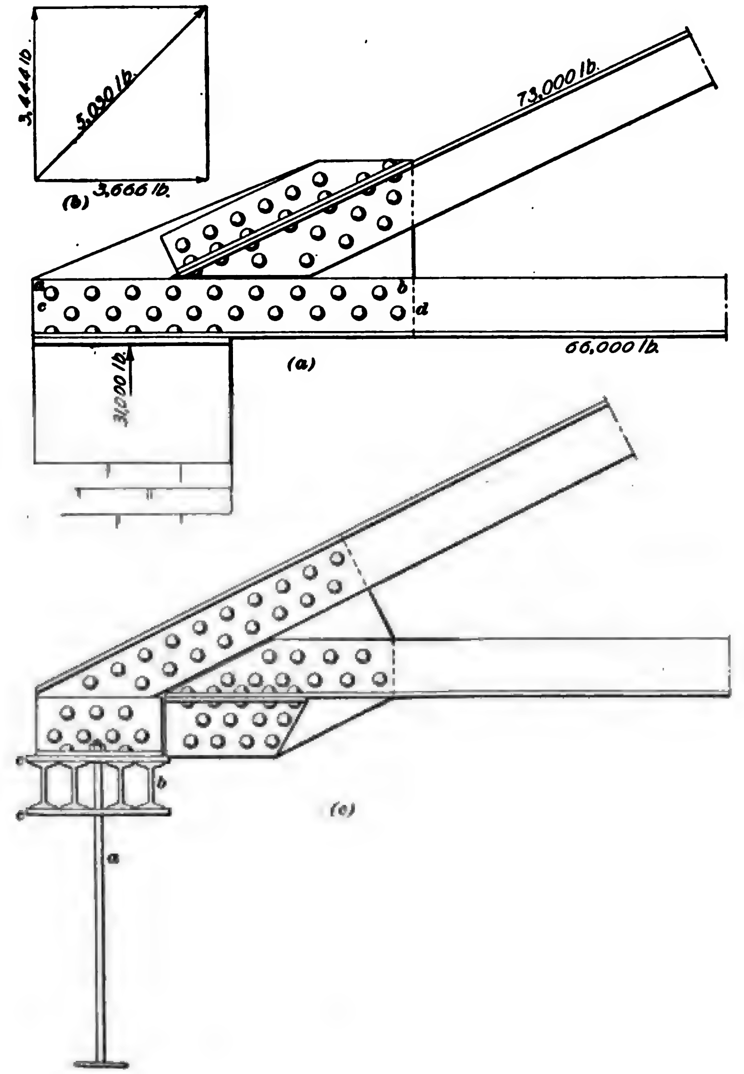


FIG 54

with the neutral axis of the chord, the gusset plate being carried out far enough to centralize the bearing. The angles of the tension member are connected to the gusset plate by one leg only, and since in this case the horizontal flanges are used for a bearing on the wall, care must be taken that the stress in the rivets due to the tension in the tie-member and the vertical reactions at the wall bearing combined do not exceed the maximum working stress of the rivets. In calculating the strength of the tension member, it must be remembered that only one leg of the angle may be considered, because the leg on which the bearing plate is riveted has no direct connection to the gusset. The following example illustrates these principles:

EXAMPLE.—If the tension in the lower chord of a steel truss is 66,000 pounds, and the reaction 31,000 pounds, what will be the greatest stress in the rivets directly over the bearing plate connecting the lower chord to the gusset plate, provided that the joint is constructed as in Fig. 54 (*a*)?

SOLUTION.—The eighteen rivets along the lines *a b* and *c d* withstand equal shares of the tensile stress of the lower chord, each providing for $66,000 \div 18 = 3,666\frac{2}{3}$ lb. The reaction is distributed among the nine rivets located along the line *c d* directly above the plate, giving each $31,000 \div 9 = 3,444\frac{4}{9}$ lb. to care for. The parallelogram of forces is laid out as in (*b*), and from this diagram each rivet immediately above the plate is found to be subjected to a stress equal to the resultant of the forces of 3,444 and 3,666 lb., which is found to be 5,030 lb. Ans.

If the rivets are $\frac{3}{4}$ inch in diameter and the gusset plate $\frac{3}{8}$ inch thick, they are amply strong, for when an allowable unit shear of 9,000 pounds and an allowable unit bearing of 18,000 pounds are assumed, the safe resistance of a steel rivet is somewhat in excess of 5,030 pounds.

103. In the heel joint in Fig. 54 (*c*) is given another method for securing a concentric bearing on the wall. The gusset plate is extended below the tie-member instead of above the rafter member as in (*a*). Small I beams are placed along the top of the wall and form a support for the truss whose bearing is constructed of a plate and angle clips. Additional anchorage is obtained for the truss by

building into the masonry a long bolt *a*, which passes between the I beams and through the angles and plates. The tension member is connected by both legs to the gusset plate and is figured for its full section, minus the rivet

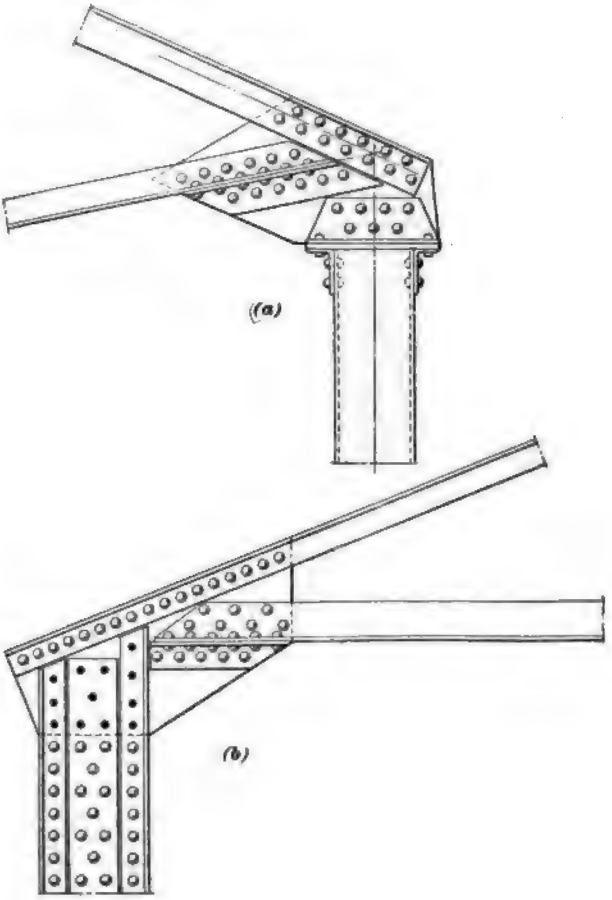


FIG. 55

holes. This design is useful where an uninterrupted line is desirable for the upper chord.

104. In Fig. 55 (*a*) is given a detail for the heel joint of a truss whose lower chord slants downwards. The truss is

connected to the top of a channel column by means of angle clips and bolts, the whole being arranged so that the center line of the column intersects the panel point. In Fig. 55 (b), the truss is connected to a column made of plates and angles. The gusset plate is placed between the angles of the column and connected thereto by means of splice plates, making a compact and rigid joint.

105. Often when a ceiling exhibiting some particular architectural effect is desired, it becomes necessary to carry the bearing below the line of the lower chord. In such cases the design in Fig. 56 is suitable. Here a large gusset

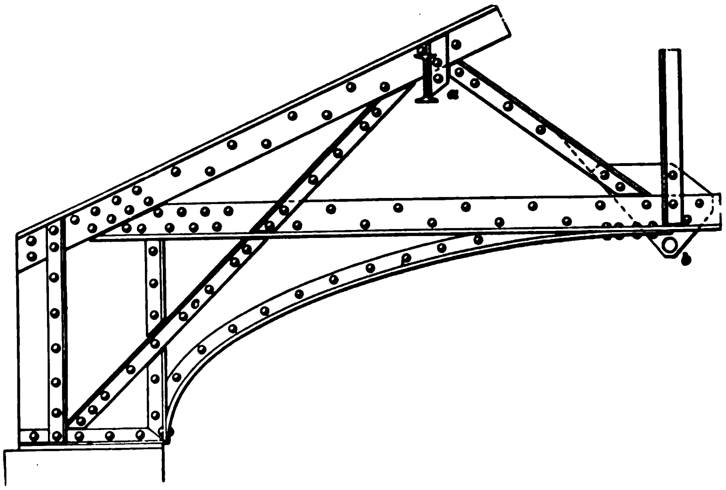


FIG. 56

plate extends almost to the first connection of the lower chord and includes the first panel point of the upper chord, forming practically a small cantilever beam at the end of the truss that would be in action more under an oblique load than under a vertical load. This detail should be analyzed for both shearing and transverse stresses. An angle brace *c* extending from the corner of the bearing to the first panel point in the upper chord greatly strengthens the design. The purlins consist of I beams hung by means of angle clips, as at *a*. The truss is designed to carry a

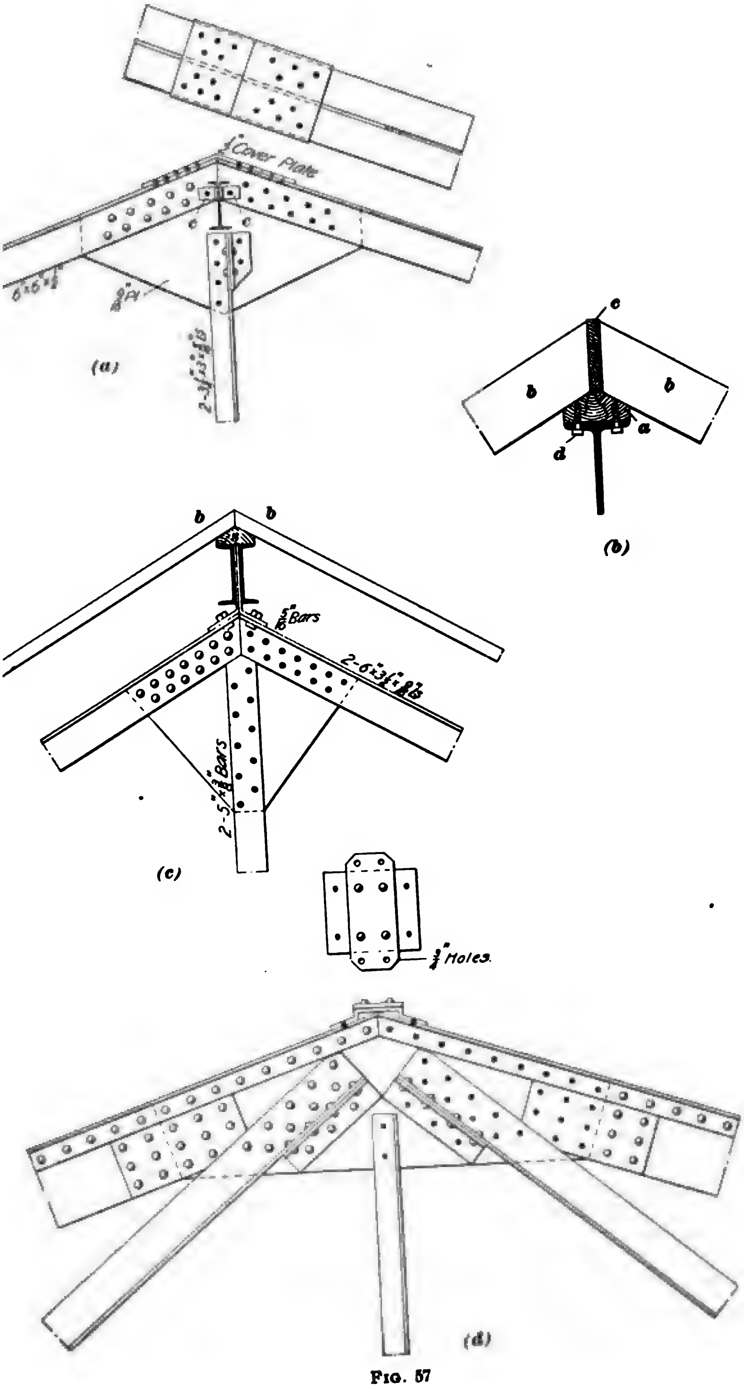


Fig. 57

suspended load from the point *b*, a hole being left in the gusset plate to accommodate the suspension rod.

106. Peak Joints.—In Fig. 57 (*a*) is given a detail of a **peak joint** where the members are connected by means of a gusset plate and a cover-plate. An I beam serves as a ridge pole, being connected to the rafter members by means of two angle clips *c*, *c*. The rafters are nailed to a strip of wood screwed on top of the I beam, as in (*b*). Another detail for a peak joint is shown in (*c*). In this case the tension members are flat bars and the ridge pole is made of two channels riveted back to back and connected to the truss by

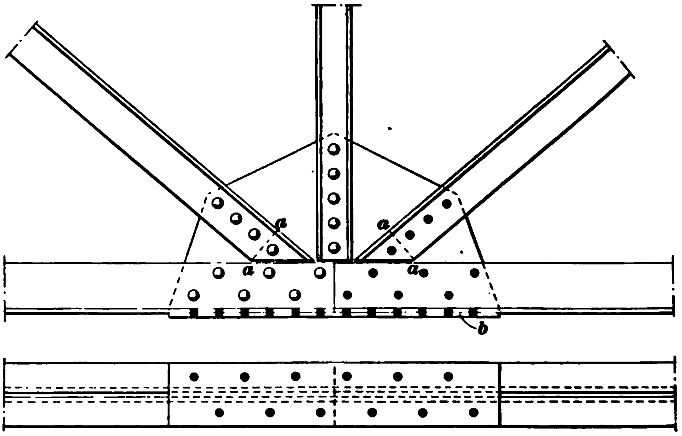


FIG. 58

means of two bent plates extending upwards between them. The roofing boards *b*, *b* are fastened to a nailing strip *s* bolted to the top of the channels. The upper chord of the truss in (*d*) is composed of two angles and a web-plate. In order to preserve the strength of the compression member, its web is connected to the gusset plate by two splice plates. The wooden ridge pole is secured in place by being bolted through holes in the bent plate that supports it.

107. Center Joints.—A very neat design for a **center joint** is given in Fig. 58, in which the gusset plate is used as a splice plate for the lower chord. In structural work the

angles are usually cut square on the ends, as shown by the dotted lines *a*, *a*, although greater neatness may be secured by cutting them obliquely, as in the figure. Such a center joint as this could be used in a structural truss constructed along the lines of a Howe truss, where the compression members are oblique and the tension members vertical. The center joints that serve also as splices are usually those at which the truss has been separated for convenience in shipment. Both field and shop rivets are used in this design, the gusset plate being riveted to the left-hand portion of the truss in the shop, and sent with that portion.

The center joint, when combined with a splice, should always be reenforced by a flange splice plate, as at *b*. Such a plate increases the strength of the splice and stiffens the truss laterally at a point that would otherwise be weak.

EXPANSION ENDS

108. When trusses of long span are exposed to great variations of temperature, provision is usually made for the expansion and contraction of the frame. This may be done by the use of roller bearings, flat plates sliding on each other, or by a swinging column or arm placed at the end of the truss.

Since flat plates are not efficient they are only used in trusses of short span whose contraction and expansion are but slight. As they are not without friction, allowance is sometimes made for a horizontal force, which is found by the following formula:

$$H = R a \qquad (6)$$

in which H = horizontal force to be introduced at expansion end of truss;

R = reaction on free end of truss;

a = coefficient of friction for metal sliding on metal.

For steel on steel or wrought iron on wrought iron a may be taken at .15, while for wrought iron or steel on cast iron a is .20.

109. In Fig. 59 is illustrated the **expansion end** commonly employed for a steel truss having a span of 60 or 80 feet. A sole plate $\frac{7}{16}$ to $\frac{7}{8}$ inch in thickness is provided at the expansion end and is incorporated in the heel joint, being secured to the angle clips with countersunk rivets, as at *a*.

An illustration of the common type of roller expansion joint is given in Fig. 60 (*a*). In this case the truss is supported on a bearing plate resting on six rollers each 2 inches in diameter and 13 inches long. The rollers are held in place by a bar-iron frame and kept the proper distance apart by separators made of pipe, through which bolts are passed

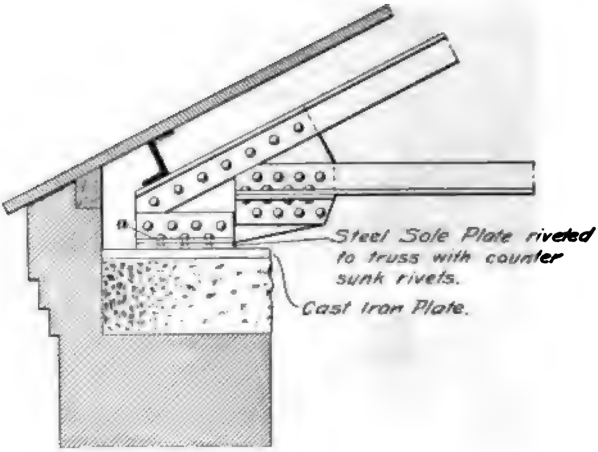
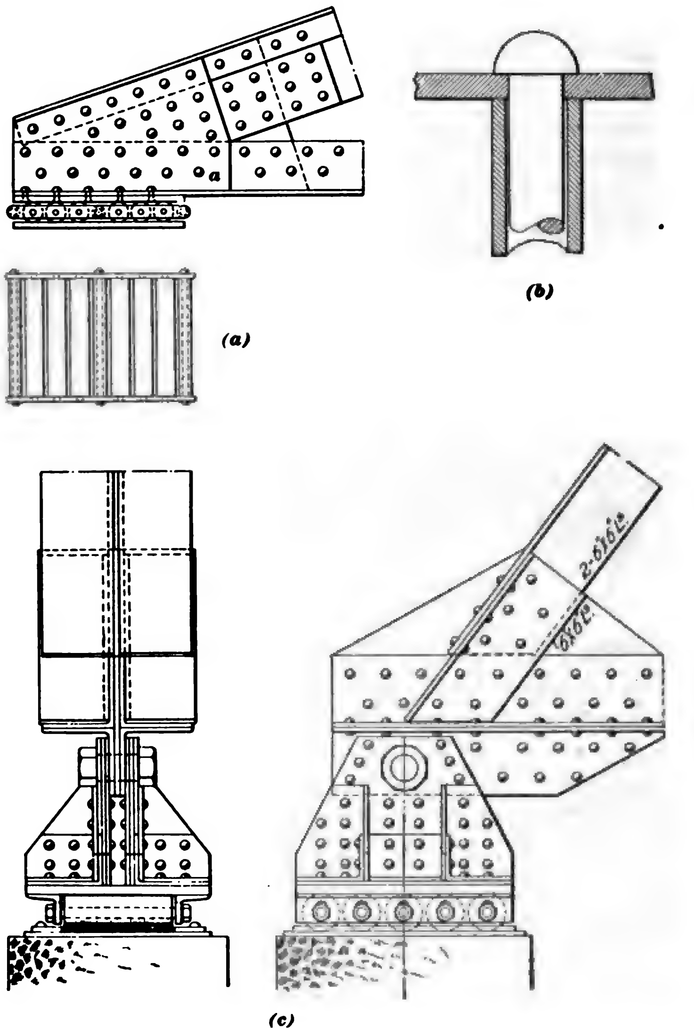


FIG. 59

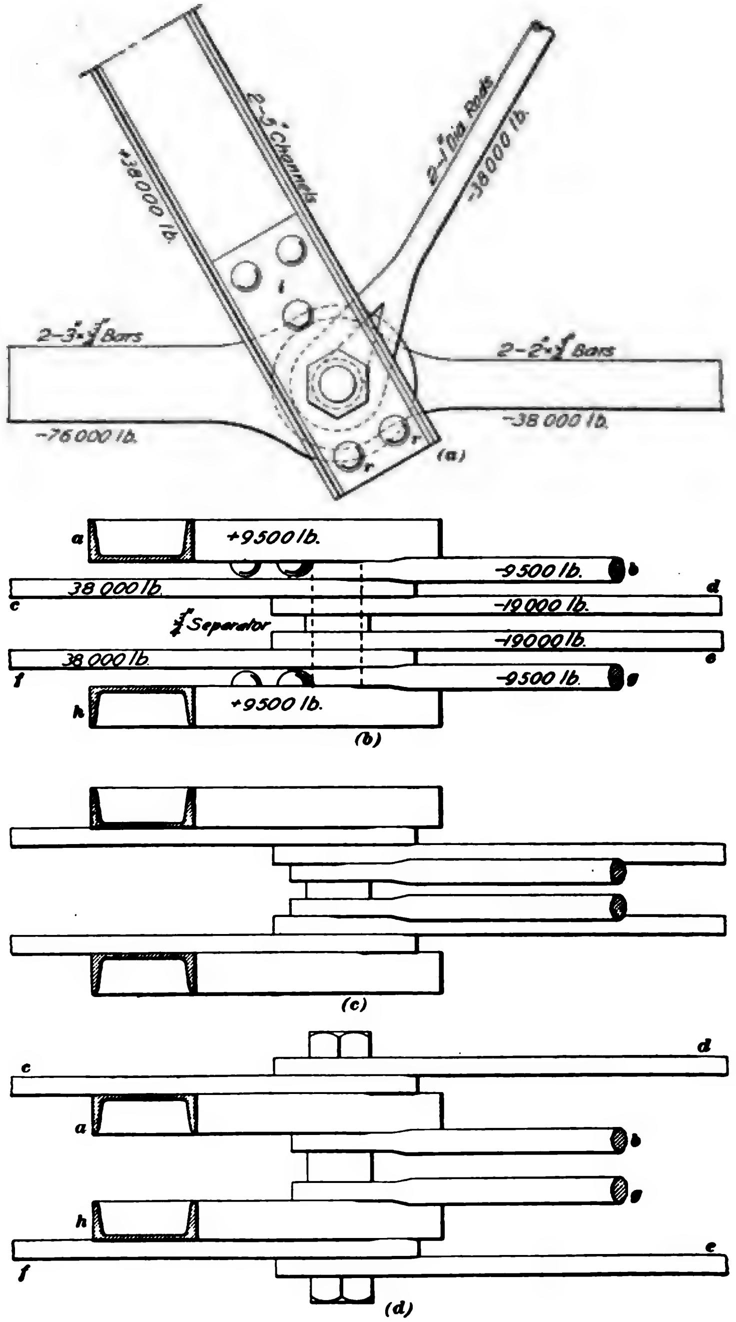
and riveted over on the ends, as shown in (*b*). The truss end is strengthened by riveting a cover-plate over both angles and splice plate, as at *a* in (*a*).

Such a roller bearing as that shown in Fig. 60 (*c*) is used only for heavy trusses with spans of 80 feet or more. It is not customary to hinge a supporting column at the top when it connects with a steel truss, but structural engineers often use a combination roller-and-hinge bearing for long trusses and heavy work. In this construction, shown in Fig. 60 (*c*), a carriage or roller-bearing supports the expansion end of the truss by means of the pin connection. By the use of



(c)

FIG. 60



Channels
+38000 lb.
Dia. Rods
-38000 lb.
Bars
-76000 lb.
-38 000 lb.
(a)
+9500 lb.
-9500 lb.
38 000 lb.
-19 000 lb.
Separator
(b)
(c)
(d)

Fig. 61

In Fig. 61 (*b*) is shown the method of arranging the several members on the pin so as to obtain the least bending moment. The two rods *b*, *g* tending to raise the pin are placed next to the strut members *a*, *h*, which tend to lower it. The tension in the two bars *c*, *f* is balanced by the tension in the rods *d*, *e*, and by the horizontal components of the stress in the members *a*, *b*, *g*, and *h*. Since the horizontal component of the stress in the members *g*, *h* is 19,000 pounds, by placing these on one side of the member *f* and placing *e* on the other side, the balance on the pin is almost perfect.

But frequently the size of the pin is not so important as it is to secure such an arrangement that the greatest possible lateral stiffness is secured. When this is the case, the members in the lower chord should be placed as far apart as possible, as shown in Fig. 61 (*c*). When all things are considered and each point given its relative importance it is found that the arrangement in (*d*) is the best that can be adopted. In this case the greatest lateral stiffness is obtained, without materially increasing the bending moment on the pin.

114. The detail shown in Fig. 62 may be adopted when it is necessary to splice the upper chord. The strut is made of two channels placed back to back, and turned so that their webs are at right angles to the plane of the truss. The pin plates are riveted to the flanges of the channels and extend up between the upper chord member, thus serving as splice plates also. These pin plates are shop-riveted to one portion of the chord member and field-riveted to the other part, and should be of such a thickness that the bearing on the pin is reduced to the allowable unit stress. The end of one of the tension rods is provided with a clevis, and the end of the other with a loop that is placed between the sides of the clevis; by this means the rods are not only connected to the pin, but are also kept in the same vertical plane.

115. The details given in Fig. 63 can be used in very large pin-connected trusses. The truss proper ends at the joints *a*, *b*, but in order to stiffen the roof against the wind a brace is extended from *a* to *c*, which increases the rigidity of

the frame. This brace must be designed to resist both tension and compression, since its stress varies as the wind shifts from one side to the other. The upper chord member is composed of two 16″ × ½″ web-plates to which four 3″ × 3″ × ½″ angles are fastened and held together by lacings running along both top and bottom of the member. Latticed beams placed centrally with the panel points, as at *d*, serve as purlins, carrying the 6-inch 14¾-pound channel rafters. The trusses are fastened to columns composed of two 15-inch

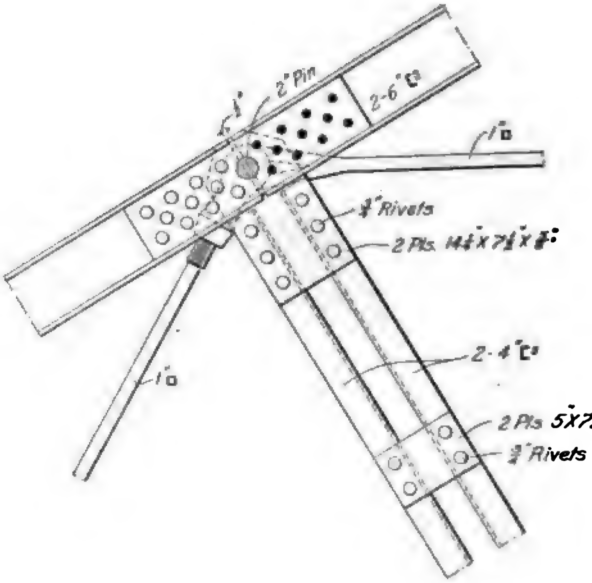


Fig. 62

42-pound I beams riveted to an 18-inch 55-pound I beam. This makes a column amply strong for the position and at the same time very simple in construction. These details are taken from the design for the Rock Island terminal station in Chicago.

116. The analysis and proportioning of an important pin connection in a roof truss are best demonstrated by considering the design of the connection and members of a joint such as is given in Fig. 64. *A B*, *B C*, *C D*, and *D A*, shown in the

frame diagram (*a*), represent members whose lengths are 23, 20, 15, and 15 feet, respectively. In the solution, all tension members are assumed to be composed of bars 5 inches

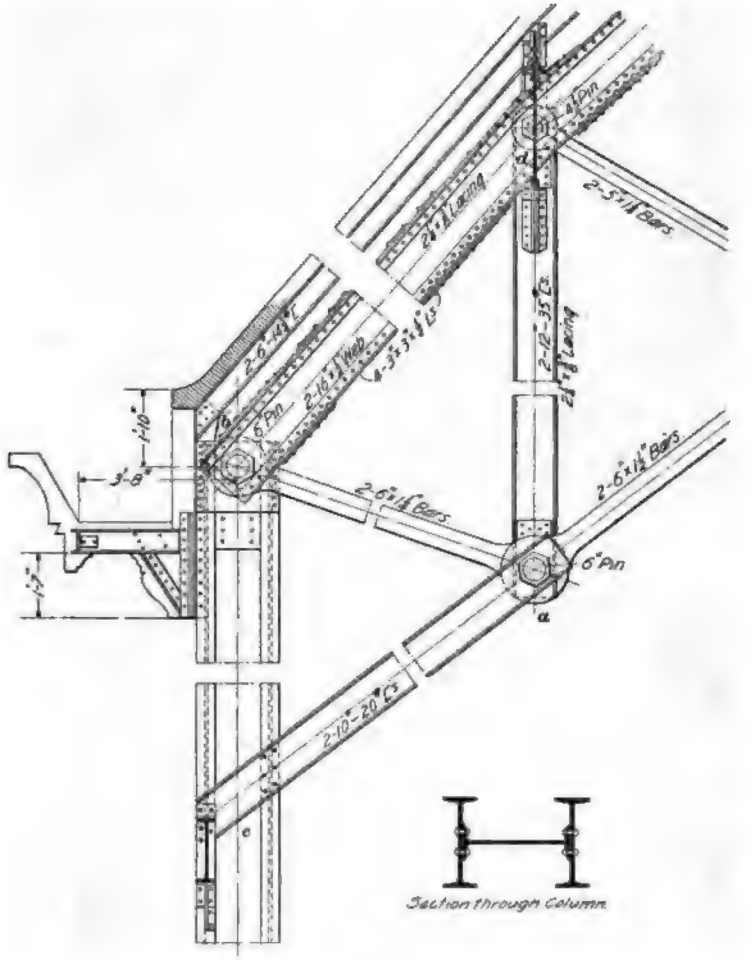


FIG. 63

deep, having a maximum allowable unit tensile strength of 18,000 pounds, while the compression member is to be constructed of channels placed back to back and latticed together.

ASTOR, LENOX AND
TILDEN FOUNDATIONS

If a factor ...
that the ...
pounds. Hence ...
is 8.82 square ...
strut is 9,790 × 8 ...
ber is required ...
it is found to be ...
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117. In prop...
example under dis...
effect of the weight ...
stress to which the ...
the deflection produ...
stress to which the ...
stress produced by ...
mula 1.

Applying this formul...

In proceeding with the problem, it is first necessary to lay out a skeleton diagram, as indicated in (*b*) by the dot-and-dash lines, on which the design is drawn after the sizes of the different members have been determined. After this is done the stresses in the several members may be either obtained from the stress diagram or calculated by some mathematical process. In this case, the compression member BC must sustain 78,000 pounds, and if the column is composed of two 9-inch 15-pound channels, their combined area will be 8.82 square inches and the radius of gyration of the column section about an axis perpendicular to the web is 3.4. Applying the formula $u = \frac{50{,}000}{1 + \frac{l^2}{18{,}000\, r^2}}$, in which u equals the ultimate strength in pounds per square inch of section for a column hinged at the ends and built of medium steel, the calculations may be made as follows:

$$u = \frac{50{,}000}{1 + \frac{(240)^2}{18{,}000 \times (3.4)^2}} = 39{,}160 \text{ pounds}$$

If a factor of safety of 4 is assumed, the safe unit stress that the column or strut will sustain is $39{,}160 \div 4 = 9{,}790$ pounds. Hence, if the area of the column or strut section is 8.82 square inches, the safe sustaining power of the strut is $9{,}790 \times 8.82 = 86{,}348$ pounds. But since the member is required to sustain a load of but 78,000 pounds, it is found to be large enough and the section originally assumed may be used.

117. In proportioning the tension members in the example under discussion, it is advisable to investigate the effect of the weight of the member and to determine the unit stress to which the member will be subjected on account of the deflection produced by its own weight. The additional stress to which the member is subjected by the transverse stress produced by its weight may be determined by formula **1**.

Applying this formula for a steel tension bar of the

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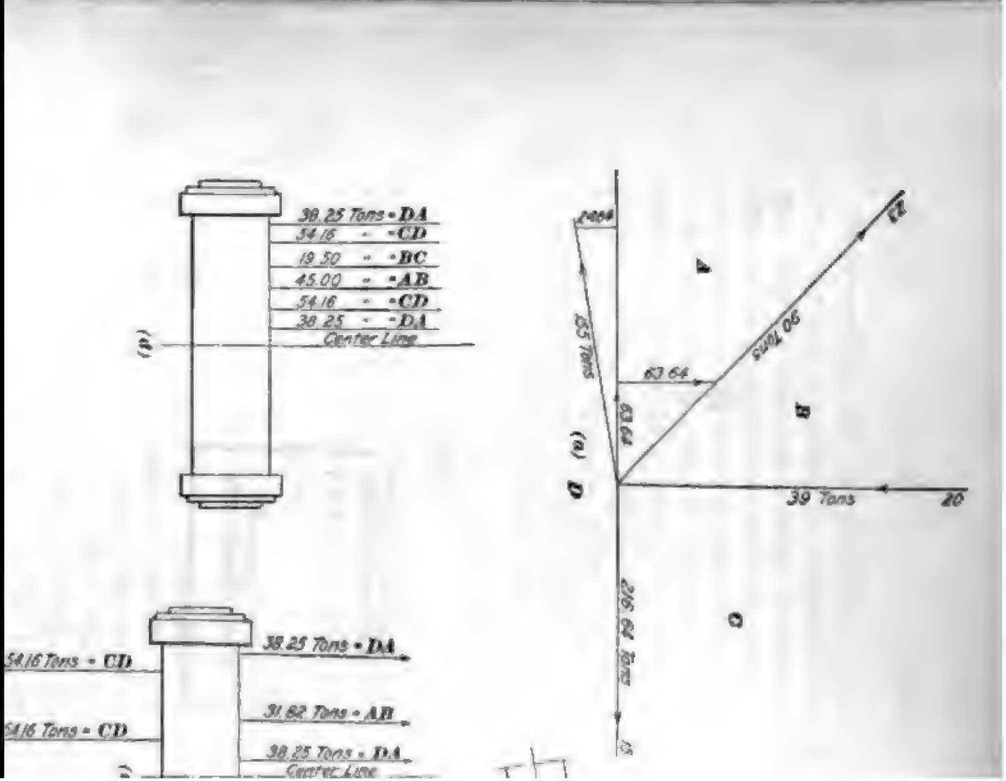
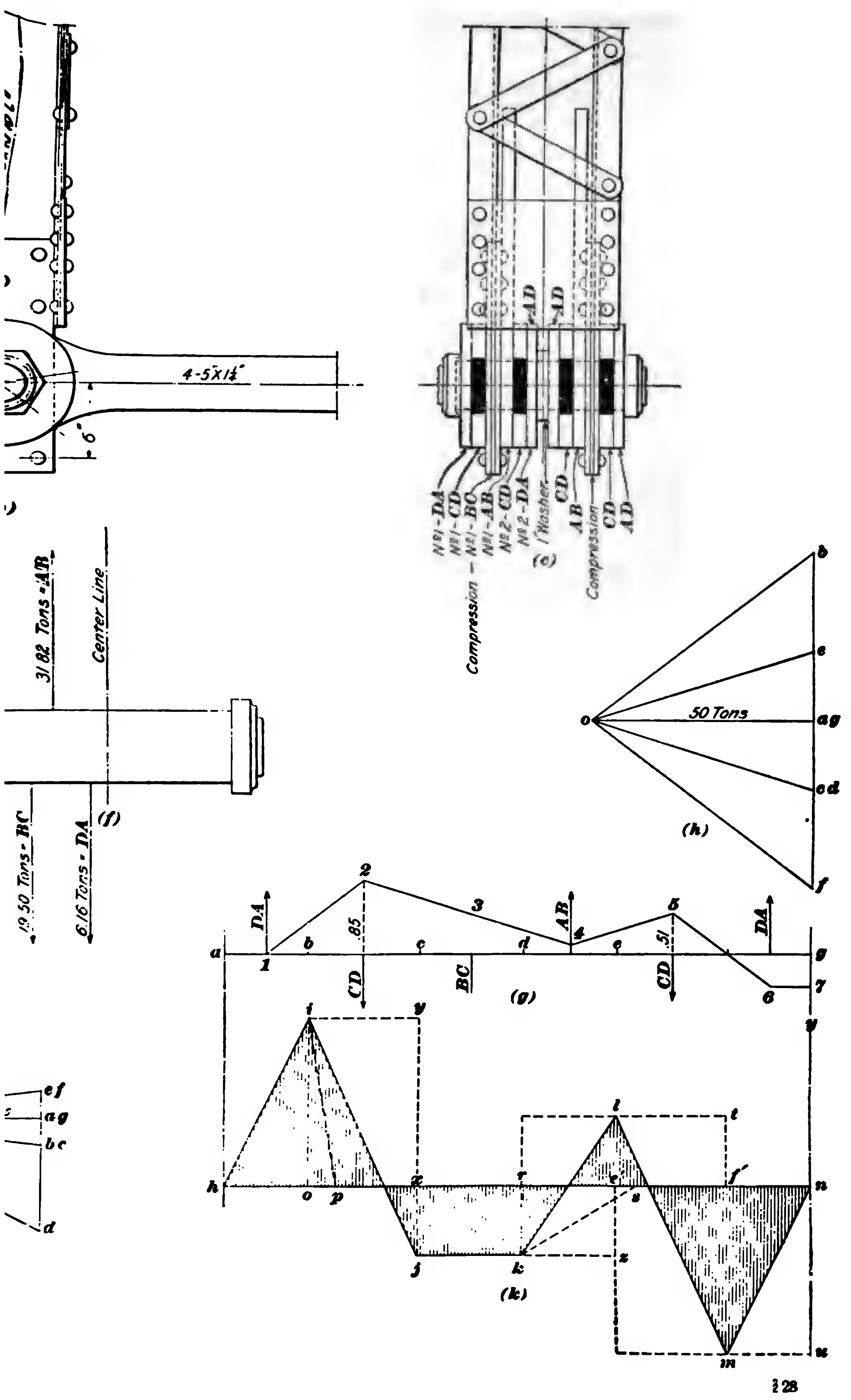


38.25 Tons = DA
54.16 " = CD
19.50 " = BC
45.00 " = AB
54.16 " = CD
38.25 " = DA
Center Line
(d)
155 Tons
63.64
63.64
90 Tons
39 Tons
216.64 Tons
A
B
C
D
(a)
23
20
15
54.16 Tons = CD
54.16 Tons = CD
38.25 Tons = DA
31.82 Tons = AB
38.25 Tons = DA
Center Line



4-5"X1¼"
6"
31/82 Tons - AB
Center Line
19.50 Tons - BC
6.16 Tons - DA
(f)
ef
ag
bc
d
Compression — Nº1-DA
Nº1-CD
Nº1-BC
Nº1-AB
Nº2-CD
Nº2-DA
Washer
CD
AB
Compression
CD
AD
AD
AD
(e)
50 Tons
(h)
(g)
(k)

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assumed width and depth of 1 inch and 5 inches, respectively, and a length of 15 feet, or 180 inches,

$$s_1 = \frac{M_1 c}{I + \frac{W_t l^2}{10 E}} = \frac{\left(.28\, b\, d\, l \times \frac{l}{8}\right) \times c}{\frac{b d^3}{12} + \frac{W_t l^2}{10 \times 28{,}000{,}000}}$$

and, by substitution, after W_t has been assumed to be equal to the strength of a 1-inch × 5-inch bar with a unit stress due to the direct tension of 17,000 pounds per square inch, or a total stress of 85,000 pounds, the value

$$s_1 = \frac{.28 \times 1 \times 5 \times 180 \times \frac{180}{8} \times \frac{5}{2}}{\frac{1 \times 5 \times 5 \times 5}{12} + \frac{85{,}000 \times 180 \times 180}{10 \times 28{,}000{,}000}} = 700 \text{ lb., approx.}$$

If the allowable direct tensile stress is 17,000 pounds, the addition of the unit stress of 700 pounds due to the transverse stress created by the weight of the bar, the total unit stress on the bar, is 17,700 pounds. As steel bars used in such structures as roof trusses can be depended on to withstand a unit stress of 18,000 pounds, it is evident that the safety of the bar is not endangered by its weight, and that a unit tensile stress of at least 17,000 pounds may be assumed in proportioning the tension bars; in fact, a unit resistance of 18,000 − 700 = 17,300 pounds may be considered as the allowable unit resistance of the bars to tension. Tension members composed of bars should be proportioned to withstand the maximum stress to which they will be subjected, and should be assumed to be of some uniform width that will permit the formation of suitable eyes at the ends of the bars. After deciding what width is to be used, the total width of bar required to resist the stress may be found by dividing the maximum stress in the member by the allowable stress for a bar of unit width, as just determined, after allowing for the stress created by the weight of the bar itself. For instance, the stress in the member *CD*, Fig. 64 (*a*), is 433,280 pounds, so that if a bar 1 inch in thickness and 5 inches deep will sustain 5 × 17,300 = 86,500 pounds, to withstand 433,280 pounds, a bar 433,280 ÷ 86,500, or practically 5 inches

...rs are used ... tches and a ... are subjected ... med width of ... required, and if ... be about 13/16 inch wide ... same manner it may ... ample for the member AB.

...ensions of the several mem... size of the pin required ... erent members on the pin must ... heoretical standpoint, it is best to ... opposing forces as near each other ... reduce the bending moment. To ... the pin, the force CD must be equal ... of the horizontal components of AB ... must be equal to the algebraic sum of the ... of AB and DA. By the method of ... forces, AB and DA are found to have hori... components of 63.64 tons and 153 tons, respectively, ... components of 63.64 tons and 24.64 tons, respect... as the algebraic sum of the horizontal forces ... 63.64 = 216.64 tons, and the algebraic sum of the ... components is 63.64 − 24.64 = 39 tons, the pin is ... equilibrium.

Now, if a combination of opposing forces can be found by which the forces in each of the four bars CD are taken up directly by one or two of the bars in the members AB or DA, the bending moment will be greatly reduced. A good arrangement is shown in Fig. ..., in which No. 1 DA is placed on the outside, then No. 1 CD, and next the vertical strut No. 1 BC, which gives ... width between the channels. These are followed ... No. 1 AB, No. 2 CD, and No. 2 DA in the order ... that CD is placed between the ties DA and ..., making a practically balanced load on ... central members DA are separated by ..., which holds all

n the one pin and facilitates painting. on each side of the washer should be together as possible; to accomplish this, cts are required in the pin plates on the he web is too thin to permit countersinking, te is required on each side of the web, the g cut to allow these tension members to set it.

4 (*d*) is a general diagram showing the location and nts of the forces that act on one-half the pin, while and (*f*) are the diagrams for the horizontal and vertical forces, respectively, giving the amount and location of each and also their direction with respect to each other. The forces *CD* oppose the horizontal forces *DA* and *AB*, as shown in (*e*), while in (*f*) the vertical force *AB* opposes the forces *DA* and *BC*. It will be noted in both (*e*) and (*f*) that the forces on one side of the pin exactly balance those on the other side.

119. The horizontal and vertical bending moments of the various forces must now be found; the maximum bending moment can then be readily obtained, since in each case it is equal to $\sqrt{M_h^2 + M_v^2}$, in which M_h represents the horizontal bending moment and M_v the vertical. When there are but two forces this is readily accomplished, but when there is a number of forces meeting at a point, it is well to use a graphical method of calculation.

In Fig. 64 (*g*) is drawn a diagram representing the resistance of the pin to the horizontal components of the forces, only one-half of the pin being shown. Beginning at the left, a line *a b* is laid off representing $\frac{15}{16}$ inch, the thickness of *No. 1 D A*; then *bc*, representing the thickness of *No. 1 CD*, $1\frac{1}{4}$ inches; *c d* representing the thickness of the member *B C*, or .29 inch for the channel and .875 inch for the two $\frac{7}{16}$-inch plates used, making a total of 1.165 inches. Continuing, *de*, *ef*, and *fg* are laid off to represent the thickness of *No. 1 A B*, *No. 2 CD*, and *No. 2 DA*, or $1\frac{1}{16}$ inches, $1\frac{1}{4}$ inches, and $\frac{15}{16}$ inch, respectively.

in width will be required. Hence, if four bars are used for this member each will have a depth of 5 inches and a width of $5 \div 4 = 1\frac{1}{4}$ inches. As the rods DA are subjected to a tensile stress of 310,000 pounds, a combined width of bars of $310,000 \div 86,500 = 3.58$ inches will be required, and if this is the width for four bars, each should be about $\frac{15}{16}$ inch wide, and 1 inch in thickness. In the same manner it may be found that two $1\frac{1}{16}$-inch bars will be ample for the member AB.

118. After finding the dimensions of the several members meeting at the joint, the size of the pin required and the arrangement of the different members on the pin must be determined. From a theoretical standpoint, it is best to place members that supply opposing forces as near each other as possible, in order to reduce the bending moment. To produce equilibrium on the pin, the force CD must be equal to the algebraic sum of the horizontal components of AB and DA, and BC must be equal to the algebraic sum of the vertical components of AB and DA. By the method of resolution of forces, AB and DA are found to have horizontal components of 63.64 tons and 153 tons, respectively, and vertical components of 63.64 tons and 24.64 tons, respectively, and as the algebraic sum of the horizontal forces is $153 + 63.64 = 216.64$ tons, and the algebraic sum of the vertical components is $63.64 - 24.64 = 39$ tons, the pin is in equilibrium.

Now, if a combination of opposing forces can be found by which the forces in each of the four bars CD are taken up directly by one or two of the bars in the members AB or DA, the bending moment will be greatly reduced. A good arrangement is shown in Fig. 64 (*c*), in which *No. 1* DA is placed on the outside, then *No. 1* CD, and next the vertical strut *No. 1* BC, which gives the required width between the channels. These are followed by *No. 1* AB, *No. 2* CD, and *No. 2* DA in the order mentioned, so that CD is placed between the ties DA and AB that oppose it, making a practically balanced load on the pin. The two central members DA are separated by a 1-inch washer, which holds all

Before completing (*g*) the diagram in (*h*) is drawn. The load line is first drawn beginning at *a*; *a b* represents the magnitude of *No. 1 D A*, as shown in (*e*), or 38.25 tons; *b c*, *No. 1 C D*, or 54.16 tons. The load next in order on the pin is *B C*, but as this is a vertical load it is not considered in the diagram for the horizontal forces, and hence the points *c* and *d* coincide. Then from *d*, *d e* is drawn representing *A B*, or 31.82 tons; *e f*, *No. 2 C D*, or 54.16 tons; and last *f g*, *No. 2 D A*, or 38.25 tons. From *a*, the middle point of *b f*, a horizontal line *o a* is drawn and some value assumed for it, as 50 tons; then the point *o* is connected with the various points on the line *b f*. Reverting to the diagram (*g*), *a 1* is drawn parallel to *o a*. A line from *1* is drawn parallel to *o b* and prolonged until it intersects a vertical line drawn through the middle of $b c_1$ at some point *2*; from *2*, a line is drawn parallel to *o c* and would intersect a vertical line through the middle of *c d* at some point *3*, but since *c d* represents the width of *B C*, a vertical member, it is not considered and the line *2–3* is prolonged until it intersects a vertical line through the middle of *d e* at *4*. From *4*, a line is drawn parallel to *o e* and intersects the vertical line drawn through the middle of *e f* at *5*. From *5*, a line is drawn parallel to *o f* and prolonged until it intersects a vertical line through the middle of *f g* at some point *6*; and from *6*, a horizontal line is drawn until it intersects the center line of the pin *x y* at *7*. Now in each case the horizontal bending moment of any member is represented graphically by the amount the broken line deviates from the horizontal at the middle of the line representing the thickness of that member, and the greater the deviation the greater is the bending moment. From this it is evident that the greatest bending moment is at the point *2*. The intensity of this bending moment is found by multiplying the amount of this deviation by the polar distance *o a*, giving a result of $.85 \times 50 = 42.5$ inch-tons, or 85,000 inch-pounds, for the point *2*, and $.51 \times 50 \times 2{,}000 = 51{,}000$ inch-pounds for the point *5*.

The vertical diagram is laid out in practically the same

way, and on the load line, *a b*, *c d*, *d e*, and *f g*, represent the amounts and directions of *No. 1 D A*, *No. 1 B C*, *No. 1 A B*, and *No. 2 D A*, respectively. The diagrams (*i*) and (*j*) are drawn similar to (*g*) and (*h*) and the deviations of the broken line from the horizontal at the points *2* and *5* are found to be .15 inch and .73 inch, respectively, making the bending moments 15,000 inch-pounds and 73,000 inch-pounds, respectively. Then, the maximum bending moment at the point *2* equals $\sqrt{M_h^2 + M_v^2}$ or $\sqrt{(85{,}000)^2 + (15{,}000)^2}$ = 86,313 inch-pounds, and at point *5*, $\sqrt{(51{,}000)^2 + (73{,}000)^2}$ = 89,050 inch-pounds. The latter is therefore the maximum bending moment on the pin, and the size of the pin may now readily be determined by substituting in the formula $M = \frac{S I}{c}$. Assuming the maximum fiber stress as 18,000 pounds and substituting $.0982\,d^3$ for $\frac{I}{c}$, $89{,}050 = 18{,}000 \times .0982\,d^3$, or $d^3 = \frac{89{,}050}{1{,}767.6} = 50.38$, and $d = \sqrt[3]{50.38} = 3.693$ inches. A $3\frac{11}{16}$-inch pin may be used with safety since its diameter, 3.6875, is only a trifle less than 3.693 inches.

120. Frequently the shear and bearing value are of as much importance as the maximum bending moment in determining the size of the pin, and hence must be investigated. Assuming a shearing value of 9,000 pounds per square inch, the total shearing value of the pin is 9,000 times the area of the pin or $9{,}000 \times .7854 \times (3\frac{11}{16})^2$, or about 96,120 pounds.

The shear is determined by drawing the horizontal and vertical shear diagrams, as shown in Fig. 64 (*k*) and (*l*).

In the horizontal shear diagram (*k*), a horizontal line *h n* is drawn, and on it the points *o*, *x*, *r*, *e'*, and *f'* are marked to correspond with *b*, *c*, *d*, *e*, and *f* in (*g*). Then through these points lines perpendicular to *h n* are drawn, on which are laid off distances to represent the stress of the various members. Thus, *o i* represents 38.25 tons, the stress in *No. 1 D A*; *y j* represents 54.16 tons, the stress in *No. 1 C D*; *z l* the stress in *No. 1 A B*; *t m* the stress in *No. 2 C D*; and *u n*

the stress in *No. 2 DA*. The line jk is parallel to hn since BC is a vertical member, and this diagram is for horizontal shear. In the same manner, the vertical shear for each member may be found as in (*l*).

In order to find the maximum shear at any point, a graphical method may be applied with advantage. Thus, the horizontal shear at the point *b*, Fig. 64 (*g*), is equal to oi as shown in (*k*), and the vertical shear for the same point is equal to oi in (*l*). Then if, in (*k*), op is laid off equal to oi in (*l*), and the line ip is drawn, this line will represent the maximum shear, as it is in reality the square root of the sum of the squares of the horizontal and vertical shears, respectively. In the same way, by laying off rs in (*k*) equal to rk in (*l*), and joining the points *k* and *s*, ks represents the maximum shear at (*d*). It can readily be seen that ip is greater than ks, hence ip represents the maximum shear and by scaling is found to be about 39 tons, or 78,000 pounds, approximately. From this it is evident that the pin is sufficiently strong in shear.

The bearing value must next be considered. The allowable tension on the bars, and consequently the bearing per lineal foot on the pin, is 86,500 pounds; hence, if the allowable bearing on the pin per square inch of section is 20,000 pounds, the required diameter will be 86,500 ÷ 20,000 = 4.325 inches; hence, a $4\frac{3}{8}$-inch pin may be used.

121. Pin Plate for Channel.—The total strength of the channel column is 86,348 pounds, and since the allowable pressure per square inch on the pin is 20,000 pounds, the required area on one side will be $\frac{86,348}{2 \times 20,000}$, or about 2.16 square inches. The total bearing of the strut on the pin is equal to the product of the diameter of the pin by the combined thickness of the pin plate and channel. This thickness for one member is equal to the sum of the width of the channel web and the two $\frac{7}{16}$-inch plates, or $.29 + 2 \times .4375 = 1.165$, and for the two members is 2.33 inches. Hence, the bearing area equals 4.375×2.33, or about 10 inches, which

is ample. Then the bearing per square inch is equal to $\frac{86,400}{2.33}$, or 37,081 pounds, and for a $\frac{7}{16}$-inch plate would be $\frac{7}{16} \times 37,081$, or about 16,200. Assuming the bearing value at 12,000 pounds per square inch, the value of a $\frac{3}{4}$-inch rivet in single shear, when the plate is $\frac{7}{16}$-inch thick, is 3,940. Hence, $16,200 \div 3,940$, or 5 rivets will be required, but to make the design symmetrical seven rivets have been used, as shown. The pin plates extend 6 inches beyond the center line of the hole and are made as wide as the channel flanges will permit. To bind the whole together two rivets are placed through these plates beyond the hole.

The required outside diameter of the eyes of the tension bars is $4.375 + 1.4 \times 5 = 11.375$ inches. From these dimensions the figure may be completed as shown.

ROOF-TRUSS DESIGN

(PART 2)

ASSEMBLED DESIGN

INTRODUCTION

1. This Section, which treats of the assembled design of roof trusses and frames, is especially valuable to the student, for in nearly every instance the illustrations are actual working drawings that have been studied in the offices of practicing architects or engineers, for symmetry of appearance, strength, and economy of construction. The text matter is purely descriptive, and is intended to explain the various features of the submitted design, but the student is particularly requested to observe carefully the details of the drawings and illustrations, and should, if time will permit, lay out these drawings to a large scale and make the necessary calculations for the design of the connections.

2. When the form of the truss has been decided on, and the stress diagram has been worked out and completed, a general design giving the sizes of the members in the truss is made, and from this the details may readily be designed. The details of the truss connections and the assembled design of the several parts require very careful study by the engineer. When the details have been roughly sketched and carefully studied, a finished drawing of the truss may be made, with all the notes and data necessary to explain every feature of the construction. Sometimes, in making the drawings of the roof trusses, two scales are used, a small one for

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laying out the outlines of the truss, and a larger one for the details of the joints and portions of the truss that require particular explanation. If the details were shown on the general drawing of the large trusses, a scale of sufficient size to give an accurate idea of the details would make the drawing of such size as to be inconvenient. For the general drawing, a scale of $\frac{1}{8}$ to $\frac{1}{4}$ inch to the foot is adopted, while for the details it is customary to use a scale of $\frac{3}{4}$ inch to the foot. On the general outline diagram, the direction and relation of the members to each other, their lengths, and the distance between panel points, as well as the cardinal measurements of the truss, are given, and frequently the sizes of the members are shown. On the detail drawing, all the information relative to the depth of the cuts and the size of washers in timber trusses, also the pitch of rivets and size of gusset plates in steel trusses, are given.

In laying out the general drawing of roof trusses and frames, there are many questions to be decided; one of the most important relates to the shipment of the truss. It is evident that, particularly in steel trusses which are mainly built in the shop, the available means of transportation will only accommodate certain sized pieces, depending on the length of the cars and the height of the tunnels through which they are to be taken by rail, or the dimensions of the hatch if sent by water. In wooden trusses, the material is usually sent direct to the field, where the entire truss is constructed and put together. In making the assembled and detailed drawings of steel trusses that are to be shipped by rail, it is very desirable to know the maximum height of load that can be handled by the railroads. This varies to some extent with the different roads, as it depends on the clearance of the bridges and tunnels. Usually the height of a tunnel or cross-bridge is not under 16 feet 6 inches. It is desirable to make the sections of such size as to be readily transported to their destination. The usual rule, however, is not to make any piece of a greater height than 11 feet. In riveted truss work, it is advantageous to build the truss in as few pieces as possible, so as to obtain

the advantage of a maximum amount of shop work and a minimum amount of field work.

The question of erection also enters into the problem of designing a truss; that is, it must be decided whether it would be better to build the truss on the ground and raise it in one piece, or to build it in place, supporting its weight on false work or by means of poles and derricks. Considering the design from the latter standpoint, some knowledge must be had of the strength of the derrick available. For instance, if the truss should weigh 25 tons, and it was shown that the only derrick available had a capacity of 10 tons, it would be necessary to divide the truss so that the rigging apparatus would not be overloaded.

TIMBER TRUSSES

3. Simple Trusses.—Roof trusses are seldom built for spans less than 20 feet, and for spans up to 35 feet the simplest form, Fig. 1, is all that is usually required. In the construction of this truss, no iron tension rods are used,

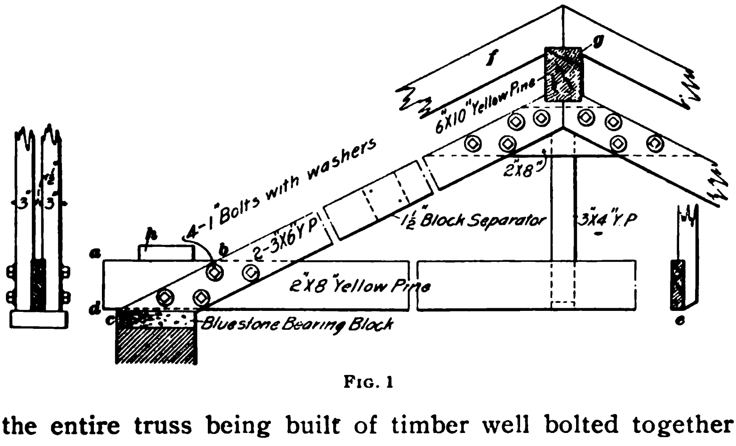

Fig. 1

the entire truss being built of timber well bolted together at the principal joints or connections. The tie-member consists of a 2″ × 8″ yellow-pine piece, while each rafter member is composed of two 3″ × 6″ yellow-pine pieces bolted each side of the lower chord, and held together at

In order that the connection at the h
be more secure, the 2″ × 8″ yellow
notched ½ inch on each side for the p
pieces composing the rafter member
resistance to shear of twice the area a
relieves the bolts of most of the
rafter members can be carried through
of the tie-member so that their ends ma
the bluestone bearing block. Thus, a
ular to the grain is eliminated, for the
the truss is carried on the end grain o
If it is required that the tie-members sh
ceiling, the 3″ × 4″ yellow-pine suspe
introduced. This member is notched
spiked or bolted at its upper end to th
at its lower end to the tie-member.

It is advisable in roof-truss construc
never to use bolts less than ¾ in
smaller bolts are likely to bend, and
ply the strength necessary at the ro
resist the tension and twisting incide
ing. In the construction shown, it is
the roof sheathing by the secondary r
at f. These are sustained at the apex
as indicated at g, while at the eaves t
the plate h. Such a truss as this is a
of roofs over small factories, dye houses,

are constructed of 4″ × 6″ yellow-pine pieces, and the thrust or compression in the rafter member is 11,200 pounds, while its horizontal component in the tension member is 10,000 pounds. In order to resist the tension in this member, it is advisable to provide sufficient shearing strength between the points *a* and *b* to resist the entire stress. The bolt *c* is then necessary only to hold the parts securely together. The suspension rod *d* is subjected to no actual stress, but is necessary to prevent the tie-member from sagging with its own weight. The rod also acts as a dowel for the apex of the truss *e*, but it would be well to spike this

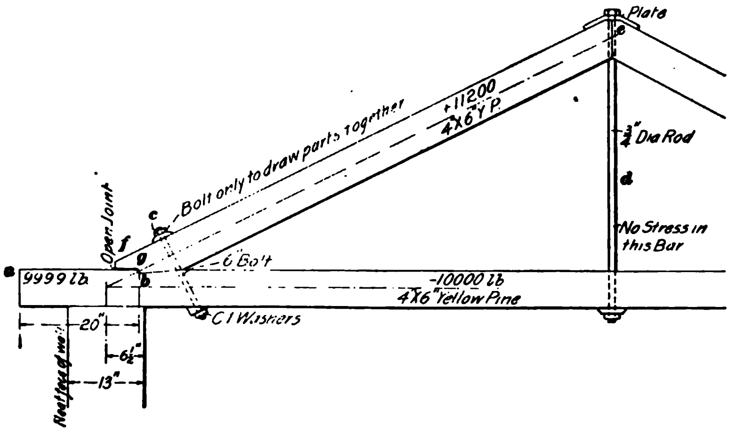

FIG. 2

joint or supply fish-plates on each side, with through bolts. The best way to cut the heel joint of the truss is shown at *f*, although sufficient bearing area to resist the stress parallel with the grain must be secured at *b g*. It is almost useless to make two cuts at this joint on account of shrinkage, which causes one cut alone to take the entire bearing.

5. Howe Trusses.—A type of truss that is much used for factories and rough construction, and for finished work where the structural parts of the building are covered by ceiling or plaster, is shown in Fig. 3. This is the simplest form of a Howe truss, and while the details of construction

are frequently left to the carpenter, greater rigidity and economy of design may be secured by a careful study of the problem. In this truss, which is for a span of 25 feet from center to center of wall, the rafter members are composed of two 2″ × 6″ yellow-pine timbers bolted on each side of a 2″ × 6″ tie-member. The intermediate struts, which support the centers of these rafter members or resist the load on the intermediate purlins by transmitting it to the center, are composed of 2″ × 6″ yellow-pine timbers. The thrust of

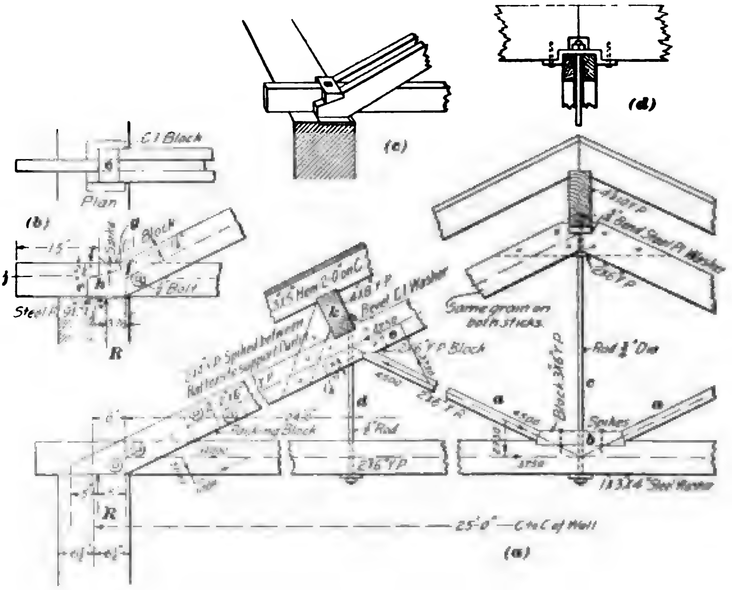

FIG. 3

the struts a, a is sustained by a block b, which is notched into the tie-member about $\frac{1}{2}$ inch, and is secured from sliding sidewise and turning by the tension rod c and the spikes, as shown. The only member that is not subjected to stress is the rod d, which is employed merely to hold the component parts of the frame together. There are few bolts in the heel connection where the rafter member joins the tie-member, and therefore resistance must be

obtained by gaining the rafter member into the tie-member about $\frac{1}{4}$ inch, or by some such expedient as that shown in (*b*). In this construction, the ends of the rafter members are cut as shown at *e f g*, and the tie-member is cut as illustrated by *g f h i*. These cuts provide an opening, into which may be inserted a cast-iron block that is held in place by spikes. This detail of construction is unique, and is entirely practical; by means of it, all the shearing strength of the surface of the tie between the points *h* and *j* is obtained. Besides this, additional shearing resistance may be obtained by gaining the rafter pieces into the tie-members on both sides. This construction is probably more clearly illustrated by the perspective shown in view (*c*). The intermediate purlins *k* are supported against overturning by a bracket or brace spiked between the rafter members, and are held very securely. It is necessary, in order to obtain the proper bearing for the strut piece *a*, to introduce packing pieces *e*, in view (*a*), which, together with the purlin support, act as separators. The only remaining feature that it is necessary to describe is the connection at the apex of the truss. A wrought-iron washer is bent to the shape shown in (*d*), and answers the double purpose of a washer and a tie and support for the ridge piece.

6. A larger and heavier Howe truss, which would be adequate to support the ordinary roof loads over spans from 35 to 45 feet, is shown in Fig. 4. The tie-member and rafter members, in this case, are composed of two 3″ × 8″ yellow-pine timbers. In this construction the rafter members are not extended down each side of the tie-members, but the entire truss is flush on the faces, so that it is necessary to use numerous packing pieces and separators. The heel connection of the truss is formed by making the usual cuts and providing the necessary shear between the points *a* and *b*. The truss, being of considerable span, requires a heavy tension rod in the center, and an adequate bearing is provided for the struts *c*, *c* by a cast-iron block, as at *d*. This block is so arranged that it is impossible for the struts to

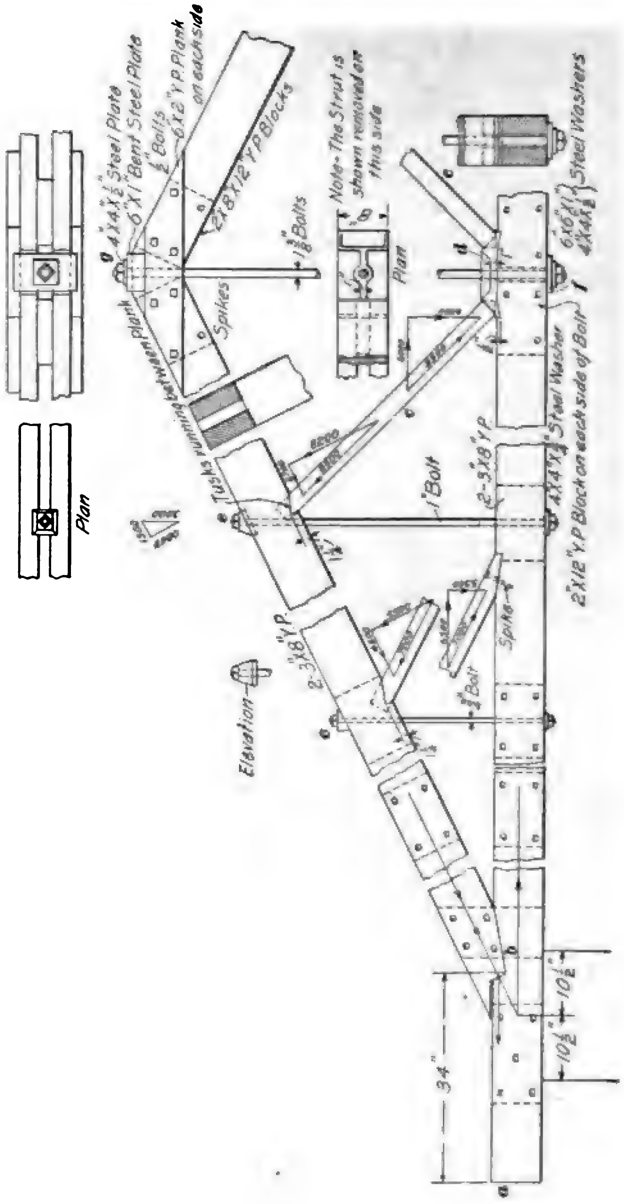
Steel Plate
Bent Steel Plate
Bolts
Y.P. Plank on each side
Y.P. Blocks
Note-The Strut is shown removed on this side
Plan
Spikes
Bolts
Tusks running between plank
1" Bolt
2-3"X8" Y.P.
Steel Washer
Y.P. Block on each side of Bolt
Steel Washers
Elevation
Plan
34"
10½"
10½"

FIG. 4

slip out of the receptacle provided for them in the casting. The upper ends of the struts *c*, *c* are mortised between the two 3″ × 8″ pieces composing the rafter members. The intermediate vertical tension rods must be provided with the necessary cast-iron angle washers *e*, *e*, and the central vertical tension rod must have adequate washers at *f* and *g*. The apex joint of the truss is formed by securely packing between the two pieces composing the rafter members, butting them and securing them with 2″ × 6″ yellow-pine clips, one on each side. Sometimes they are spiked or bolted with lagscrews or through bolts. The washer for the vertical tension rod at the top, as shown at *g*, is a U-shaped plate bent so as to further secure the rafter members against jarring or being forced out of alinement. The purlins for the support of the roof may be either carried by wrought-iron hangers or spiked to the upper surface of the rafter members. The stresses in the several struts of the frame are analyzed into their horizontal and vertical components by the force polygons shown in the figure.

7. The trusses so far described have been constructed of planking; that is, they have been built up of narrow timbers, such as are used for common joists. Frequently it is desirable to construct a truss of heavy timber, though a truss constructed of light timber, if well put together, has the advantage over one constructed of solid timber, in that the light timber is likely to be sounder and better seasoned.

The truss shown in Fig. 5 has no unusual features. The resistance at the heel is obtained by the shear at the upper side of the tension member, and the apex joint is constructed by merely butting the timbers, where they are held securely in place by spiking and are prevented from displacement by the lips *a* cast on the washer; they are further secured by the vertical tension rod. This truss is sufficiently strong for spans as great as 50 or 55 feet. The vertical tension rod is heavy, and in order to get sufficient net area it is necessary to upset, or enlarge, it on the ends so that the sectional area at the root of the threads will be in excess of the net area of

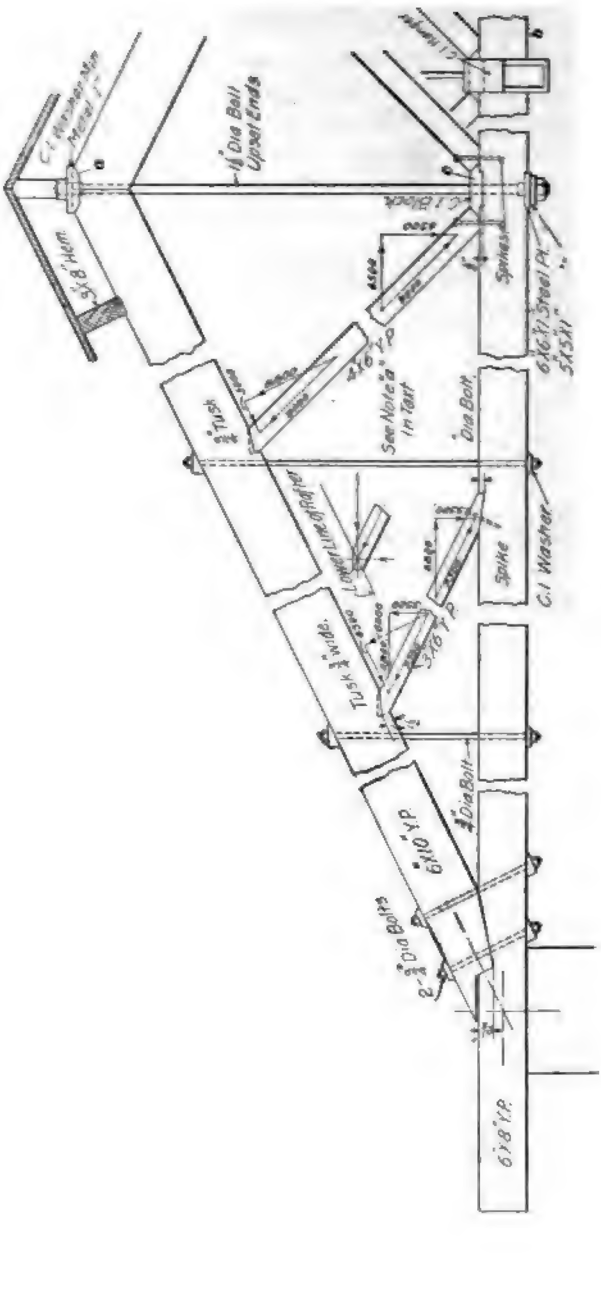
6"X8" Y.P.
6"X10" Y.P.
Dia Bolt
Dia Bolts
Tusk $\frac{3}{4}$" wide.
Lower Line of Rafter
Spike
C.I. Washer
See Note "a" in Text
Tusk
Spikes
Steel Pl.
C.I. Block
Dia Bolt Upset Ends
3"X8" Hem.

FIG. 5

he bar. This truss is flush on both faces, for the struts are inches in width, the same as the rafter and tension members. There is shown at *b* an alternate detail for the conection of the lower central joint of the truss. Such a hanger ould be used for the support of ceiling joists, and with could be combined a cast-iron block, as shown at *c*. This onstruction would not be too complicated and would be istifiable if there were a number of trusses of this size to e built. It will be noticed that in this figure, as in the revious ones, an effort has been made to bring the interection of the center lines of the rafter member and tieember directly over the center of the wall. This should be one wherever possible, for in this manner a true bearing is ecured, and the likelihood of bending stresses is eliminated.

8. Trusses With Straining Beams.—In Fig. 6 is hown a cross-section through the roof and ceiling of a uilding with a truss that is much used where it is desired o secure an open floor space or attic, as designated y *a b d c*. The disadvantage in this type of truss lies in the act that it is not stable under an unsymmetrical load, ecause the central rectangular panel is liable to disortion. For spans of medium length, such as 35 to 45 feet, his truss can be used with perfect safety, provided that oncentrated floor loads of considerable magnitude are not nsymmetrically placed in the attic. The tie-member and training beam are subjected to equal stresses, the former to tensile stress and the latter to a compressive stress. The race struts *e*, *e* have their bearing at the foot on a 6″ × 6″ butment piece, and are mortised at the upper end into the rincipal rafter. As this rafter makes a considerable angle vith the horizontal, the thrust is correspondingly small. Therefore, sufficient strength may usually be obtained at the eel by the use of the cast-iron block shown at *f*, the bolt *g* eing necessary only to hold the members in their relative ositions. In order to rigidly secure the straining beam in place, it is advisable to use bent screw plates on each side and through bolts, as designated at *h*, *h*, and also to notch

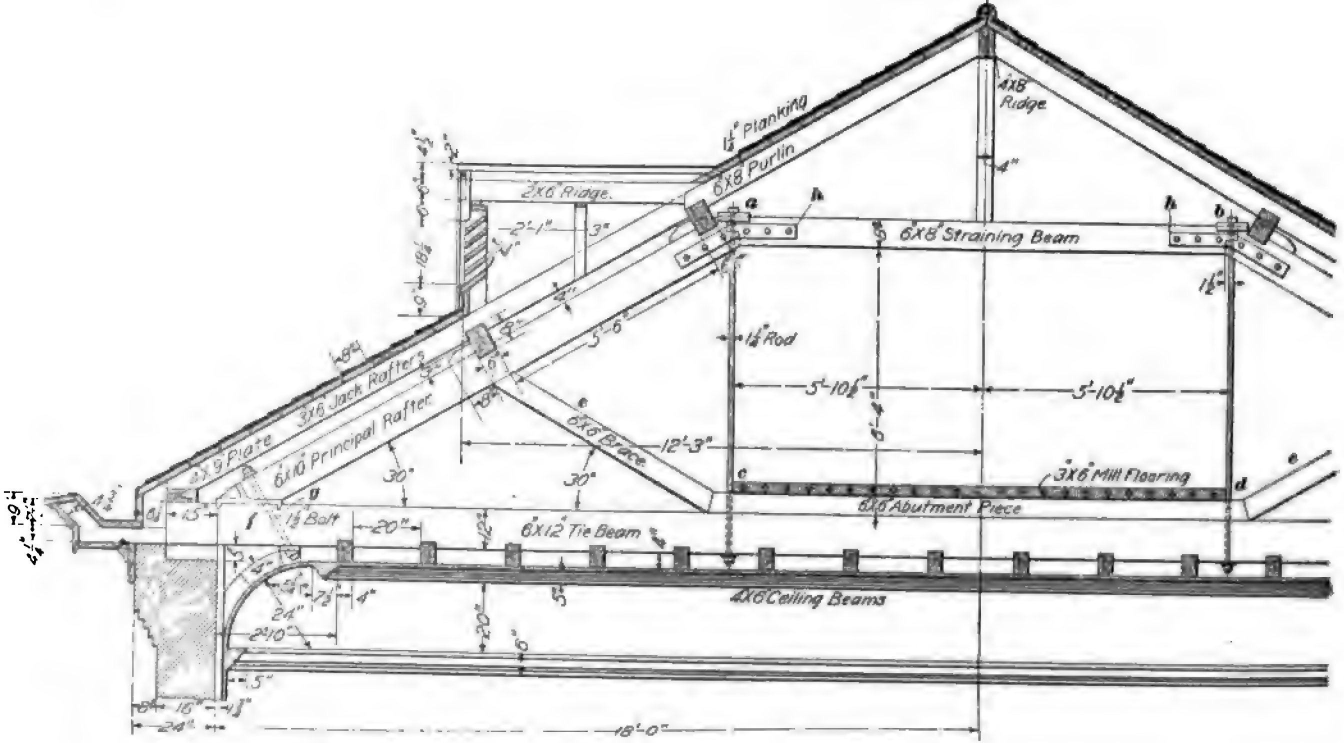
1½" Planking
6X8 Purlin
4X8 Ridge
4"
2X6 Ridge.
2'-1"
3"
6X8 Straining Beam
1½" Rod
5'-10½"
5'-10½"
3X6 Jack Rafters
6X10 Principal Rafter
6X9 Plate
6X6 Brace
12'-3"
30"
30"
3X6 Mill Flooring
6X6 Abutment Piece
6X12 Tie Beam
1½ Bolt
20"
4X6 Ceiling Beams
2'-10"
24"
18'-0"
24"
16"
5"
a
b
c
d
e
f
g
h

Fig. 6

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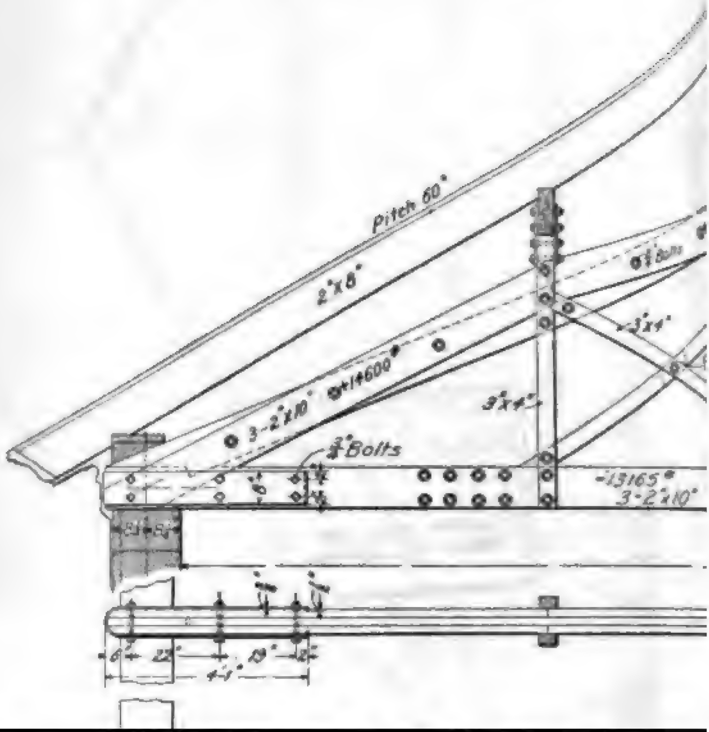
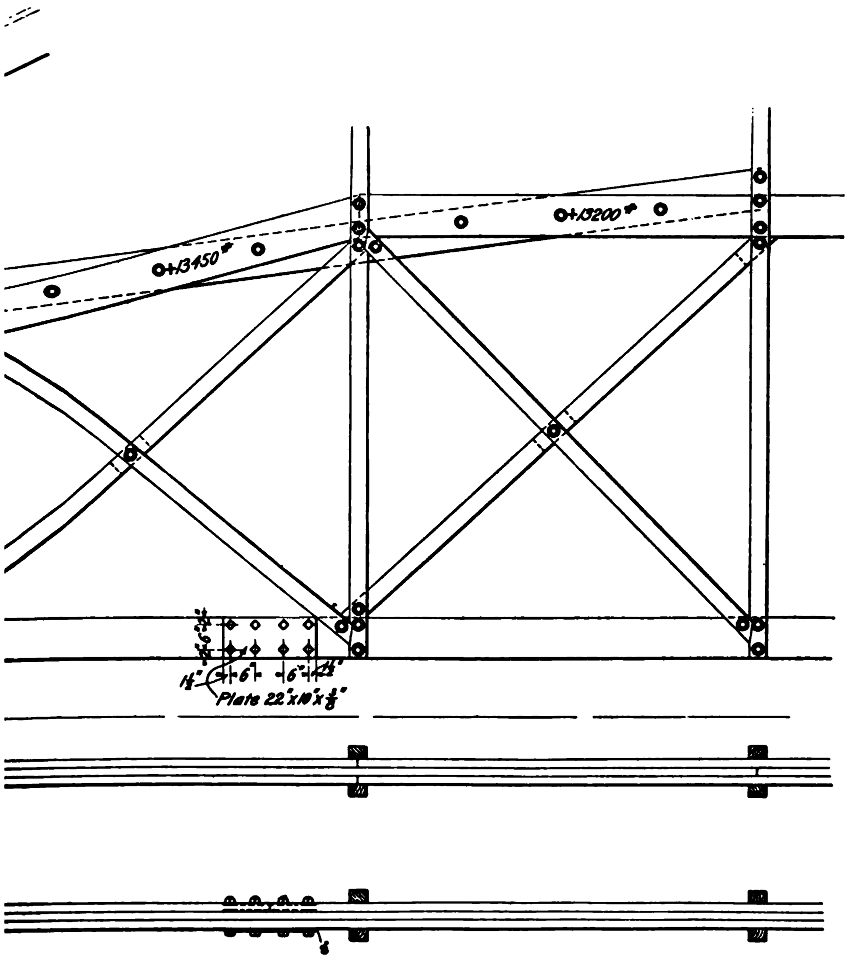
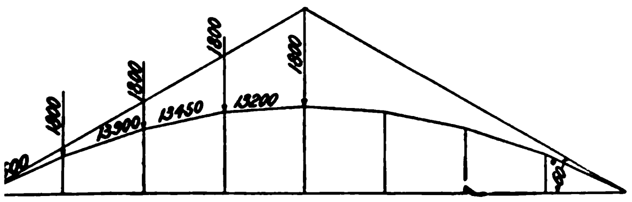
ASTOR, LENOX AND
TILDEN FOUNDATIONS



Pitch 60°
2"x8"
3-2"x10"
14600#
Bolts
13165#
3-2"x10"
3"x4"
4'-1"



...hand.



64'-0" Span-

NEW YORK
LIBRARY

the rafter members and straining beams in the manner shown at the joints *a* and *b*. Such a truss as this may be used for the assembly room of a hall, and the architectural appearance of the interior may be enhanced by a coved ceiling supported on light scantling and furring. This scantling may be fastened by wrought-iron hangers or boards spiked or nailed to the lower tie-member and to the scantling. In this construction, it is usual to employ jack-rafters supported on purlins, as shown in the figure. The distance between walls in the clear for which this truss is designed is 36 feet, though the timbers and rods are of such size that it would be amply secure for ordinary loads with a span of at least 40 feet.

9. Parabolic Trusses.—It is frequently necessary to employ lightly constructed trusses of timber for the support of roofs over car barns, repair shops, mills, and temporary assembly or convention halls. For such buildings the type of truss shown in Fig. 7 is adopted, thus facilitating the construction of the work and insuring rapidity of erection. The principle of this truss is that its upper chord follows the line of a parabola, which is the curve representing the bending moment for a uniformly distributed load. If the timber chord could be built true to the line that would represent the equilibrium polygon of the loads on the truss, theoretically there would be no need for the light cross-bracing or web members, and the component parts of the truss would be the upper chord, or arched member, and the lower chord, or tie-member.

In view of the fact that the roof may be loaded with snow on one side or be subjected to the action of the wind on one slope, and because it is impossible to distribute the dead load of the roof, and to secure actual symmetry, it is necessary to introduce light cross-bracing. In this instance only 3″ × 4″ timbers are required for the necessary cross-bracing. The upper and lower chords are composed of three 2″ × 10″ pieces securely bolted together. A careful study should be made of the splicing of each piece in the tie-member. Details

of the splicing of the pieces in this member are shown at a, b, c, and d; this splicing consists of cutting the pieces, as shown at a, and inserting the special wrought-iron tie shown at b. By using this tie, the shear of four pieces having the area $e\,f\,g\,h$, as designated at a, is secured, and additional resistance is obtained by the four bolts. All splicing of the joints in the tie-members should be broken; that is, in no instance should the pieces be spliced opposite each other, and in every instance it would be well to introduce an additional splice plate, as at i, i. A truss of this kind and of the span shown will, necessarily, be weak laterally; it is therefore necessary to introduce a system of diagonal cross-bracing between the trusses. The connection shown at the heel of the truss is peculiar, and consists of a heavy wrought-iron cleat bent around the lower chord member of the truss and let into it at the ends. In this way a great shearing area is obtained; and this shearing area must have sufficient resistance to take the entire thrust of the parabolic arch.

10. Scissors Truss.—It is frequently necessary, in order to reduce the cost of construction, to use light timbers in the construction of trusses over buildings of moderate span. In Fig. 8 is shown a church roof truss, of a type known as the **scissors truss**, with a span from outside to outside of wall of 38 feet. The truss was designed for brick walls without buttresses, and incapable of resisting any great amount of horizontal thrust; therefore, it was necessary to design it so that there would be no sagging which would cause spreading at the feet. Owing to the light construction, this truss should be used on 16-inch centers, in which case no purlins are required, the sheathing being nailed directly to the rafter members of the truss. The two compression members a are made of 2″ × 8″ spruce; these are supported some distance from the apex by the two struts h, h, which are a continuation of the tie-members b, b. These tie-members depend on the member c to supply the necessary tensile resistance, so that the two sides of the

may be considered as cambered or trussed beams ly fastened at the ends of the straining beams *h*, *h*. after members are half checked at the apex with their

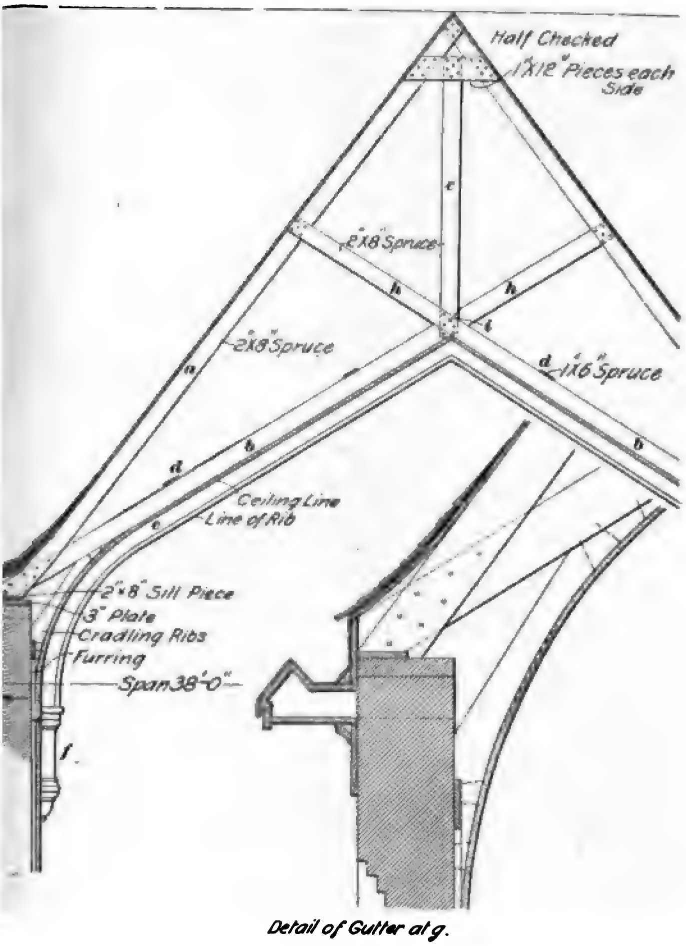


Fig. 8

s flush, and the tie-members are spiked on opposite s of the rafter member. By constructing the truss in manner, a space equal to the thickness of the rafter

member is left between the tie-members at the point *i*, and through this space is extended the vertical suspension member *c*, whose upper end is spiked to 1″ × 12″ pieces secured to the rafter members. It is evident that a roof constructed in this manner must be hidden by a plastered ceiling, the architectural appearance of which is enhanced by the introduction of a cove *e*, and the construction of false plaster ribs at intervals, as shown in the illustration. The several

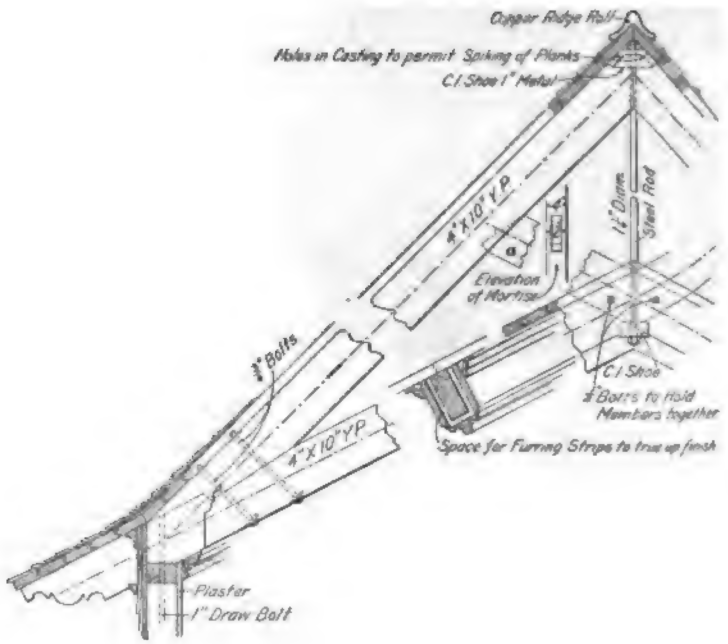


FIG. 9

details of construction at the eave line of the roof and the furring for the side walls of the ceiling is shown in the enlarged view given in the center of the figure.

11. In Fig. 9 is shown a better constructed and more finished design for a scissors truss. This was constructed for a span of about 30 feet, to be placed on 5- or 6-foot centers. The ceiling of the room is to be of natural finish, and is consequently ceiled on the upper cheek of the lower

chord, or tie-member, of the truss, this member being properly furred and finished with trim, as shown in the figure. Instead of the truss being built entirely of timber, a steel suspension rod is provided from the apex of the truss, it being noted that the washer of this rod must be set flush with the cheeks of the rafter member and provided with nail holes through which the planking can be spiked. The strut members are mortised into the rafter members with a mortise 2 inches in thickness, and though by this

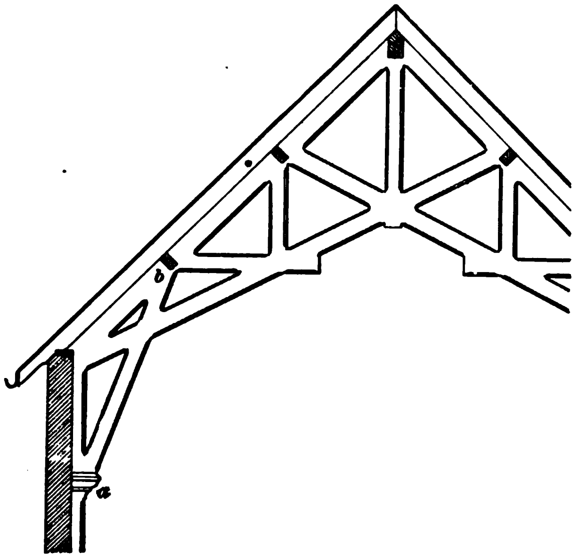


FIG. 10

means the bearing on the end of the strut is reduced somewhat, it obviates the serious cutting of the rafter members. The cut at the heel of the truss between the rafter member and the tie-member is the one usually employed for timber trusses. This illustration may be considered as an excellent design for a scissors truss, and one adapted to small buildings where some interior finish is required.

12. Types of Church Trusses.—It is not the intention in the description of Figs. 10, 11, and 12 to enter into the

details of construction, but rather to show the general outlines that are frequently employed in **church roof trusses.**

Fig. 10 shows a type of church roof truss that, while not distinctly a hammer-beam truss, follows somewhat the lines of a truss of this character. Such a truss as this would be framed together usually with heavy timbers, securely reenforced at the joints with dowels and fish-plates, and the entire truss furred and finished with neatly molded trim.

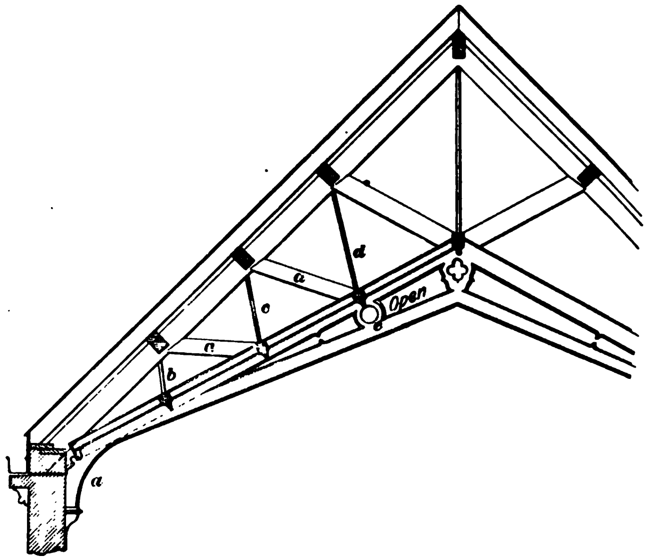


FIG. 11

Fig. 11 illustrates a truss that partakes of the nature of a Howe and scissors truss. From the illustration, it will be observed that the general outline of the truss follows a scissors type, but owing to the span it is necessary to introduce the intermediate struts *a*, *a*, and when this is done the rod *b* is used to hold the work together, while *c* and *d* are tension rods necessary for the secureness of the frame. A truss of this character probably presents the best appearance when the ceiling is carried along the upper edge of the chord member and ornamental ribs are provided, as at *e*.

The truss shown in Fig. 12 is a type of timber truss that corresponds with the Fink type. It is unusual to construct this truss of timber, but the figure shows how trusses for small spans may be built of timber on these lines. The necessary connections and rigidity at the joints are obtained by the introduction of ornamental fish- or tie-plates, as illustrated.

The true hammer-beam truss is shown in Fig. 13. This type is used in timber trusses for church construction. The truss proper extends above the line *a b*, which is the top line of the hammer beam; below this line, the construction may be considered as a brace or corbel. The upper part is

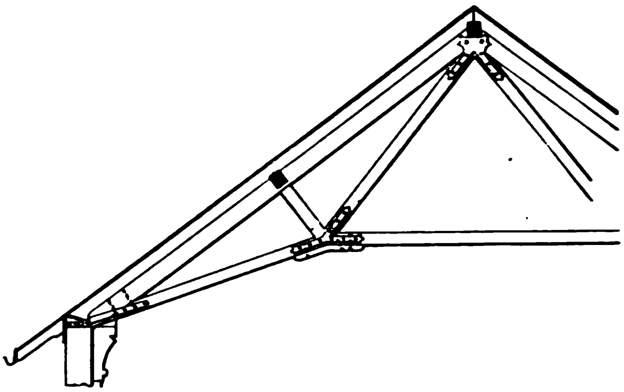


FIG. 12

designed to support three purlins and a ridge pole, each of the intermediate purlins being supported by individual struts. The members *c* and *d* must be designed for direct stress and, in addition, for a bending stress, due to the fact that their axes do not coincide with the direction of the stress. In trusses of this character, sometimes a hammer beam is made in one piece and extended from wall to wall, while it is not unusual to use a tension member between the points *g* and *h*, hiding it by ornamental turned wooden covering. It is not unusual to support hammer-beam trusses of this kind on an arcade, which separates the nave of the church from the side aisle. When this plan is adopted

the thrust at the foot of the truss can be carried back to the roof beam, as shown at *i*, which in turn transmits the thrust to heavily buttressed outside walls.

13. Gothic Trusses.—In Fig. 14 is shown a working drawing of a developed hammer-beam truss. At first glance it is not apparent that this truss is of that type, but further

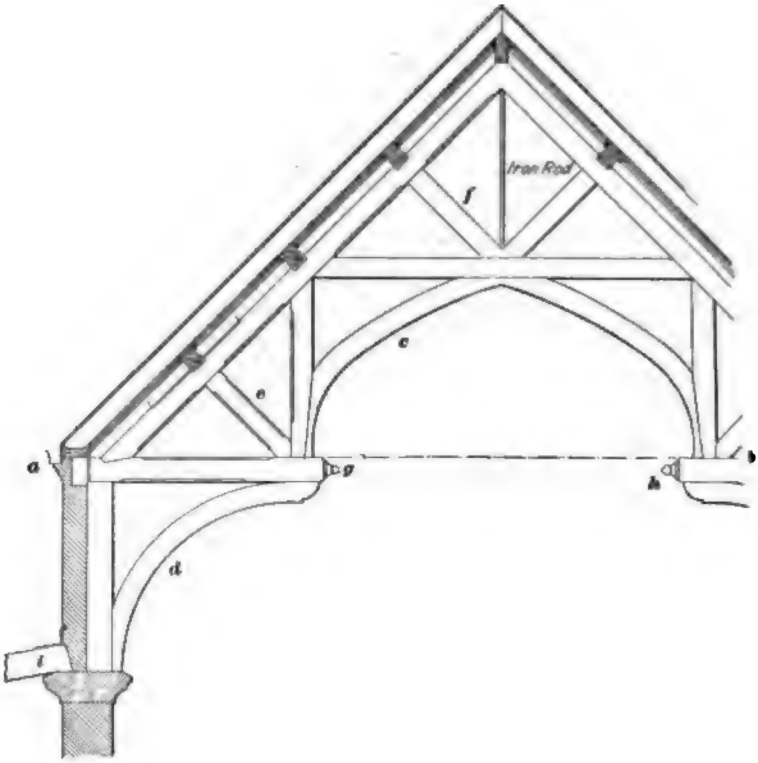
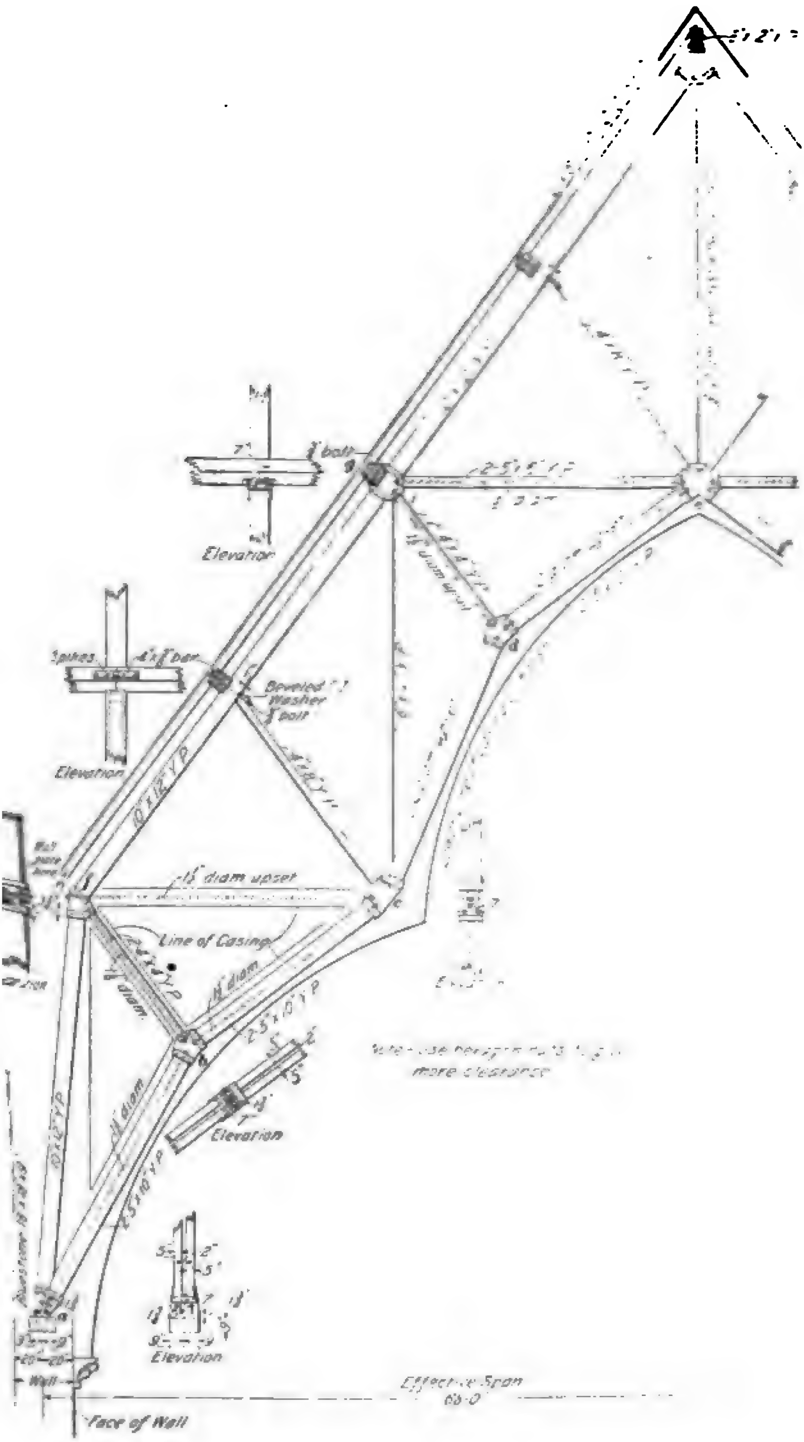


FIG. 13

study shows that the members *a b*, *b c*, *c d*, and *d e* are approximations of the curve of the typical hammer-beam truss and that the struts placed at *b f* and *d g* simply take the upward thrust of these members and thus eliminate the transverse stresses that would ordinarily occur in the curved members of a hammer beam. The truss shown in the illustration is



Elevation
Elevation
Line of Casing
Elevation
Elevation
Effective Span
Face of Wall

FIG. 11

for a span of 68 feet, which
character; its interior upper cur
arch. The form of the truss
under unsymmetrical and wind
nation of struts and tension ba
the members. These combina
small pieces of timber with a te
case of the member *c d*, whic
yellow-pine pieces with a 2-inc
and terminating in the cast-iron
of construction is excellent and
of trusses. The cast-iron boxe
desired to accommodate the m
They are arranged so as to be op
truss in order that no inconven
the bolts. In some cases, wher
do this, it may be necessary to
special form of spanner. The
should be placed centrally with
joint; and in order to facilitat
should be employed throughou

The purlins are fastened to
$\frac{3}{4}$-inch bolts that hold them firm
designed as to conveniently car
on it. Straps, or ties, are used
ing at any joint where no casting

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that is, free of columns or supports, and that the floor be close to the water level. Owing to the contour of the ground, it is necessary to use heavy buttressed walls, built as described in the notes on the drawing. The top of the wall is stepped, so that three sizes of roof truss are necessary. The trusses adjacent to the rear wall, which is built as a heavy retaining wall having counterforts on its exterior face, are built in the form shown in view (*a*). They are provided with cast-iron boxes at the apex and intermediate connection, the entire truss resting on a timber plate and being securely anchored to the wall with a 1¼-inch anchor bolt. Owing to the fact that the walls are extremely heavy and buttressed, all likelihood of a considerable horizontal thrust is eliminated, and therefore the stresses in the truss are reduced to a minimum. On account of this the vertical tension rod at the center of the truss may be reduced to the smallest diameter permissible for a frame, that is, ¾ inch to ⅝ inch in diameter. The truss marked *C* in view (*b*) must necessarily be higher, and as its cheek at the eave line must be supported, it is necessary to supply a vertical brace *h* and knee brace *i*, shown in view (*c*). At the foot-connection, as well as at the apex and intermediate joints, cast-iron boxes are used for the insertion and perfect security of the members. Where the brace *i* crosses the tie *j*, the timbers are held in position with through bolts.

In view (*d*) is shown the design of the truss adjacent to the water. At its foot is a vertical member of considerable height that is securely held against movement by means of the knee brace, as shown at *k*. Owing to the fact that the two trusses nearest to the water are of considerable height and lack rigidity in the longitudinal direction of the building, it is necessary to introduce braces between them. This lateral bracing is shown on the plan (*e*) at *l* and in position on the elevations (*d*) and (*c*).

When the general plan and layout of the building and trusses have been studied in the general drawing, it is well to investigate the details of the connections and study the special features employed in these trusses. Attention is

therefore called to Fig. 16, which shows the details for the truss *D* shown on plan (*e*), Fig. 15. The timber members of the truss are composed of 6″ × 6″ pieces, which may be of yellow pine, spruce, or fir. It is necessary to cross the two tie-members of the scissors truss under the point of the apex, and where they cross the knee brace timbers of the frame they are halved and secured with ½-inch through bolts. Intermediate purlins are placed in a vertical position and support secondary rafters spaced 2-feet on centers; these purlins are supported by lugs cast on the intermediate box connection. The several timber members butt into this box connection and are secured in place by means of draw bolts, as shown. It is necessary to provide different patterns for the cast-iron box at the intermediate purlin connection in the several trusses.

The bearing on the sill plate is obtained by making a special cast-iron support, which is spread lengthwise of the building and is well bedded and carefully designed. One of the peculiar features of the design of this timber truss is the connection at the eave line or foot of the truss. The special casting that is necessary here is so modeled as to act as a washer for a tie-bolt into the vertical member and a draw-bolt into both the vertical and main rafter members. The usual cut is made for the bearing of the rafter member on the tie-member of the scissors truss. On this detail sheet is also shown the longitudinal bracing and its several features of construction.

15. Cantilever Trusses.—In Fig. 17 is shown a type of truss that is frequently used for the support of roofs over platforms at railroad stations; it is also applicable for the construction of grandstands, where it is necessary to have a clear space in front of the spectators. Its structural elements consist of a main supporting member and the overhanging frame or truss. The roof under consideration is supported by heavy posts, which are held across the top with a heavy timber, the rectangular frame thus formed being securely held against lateral distortion by knee

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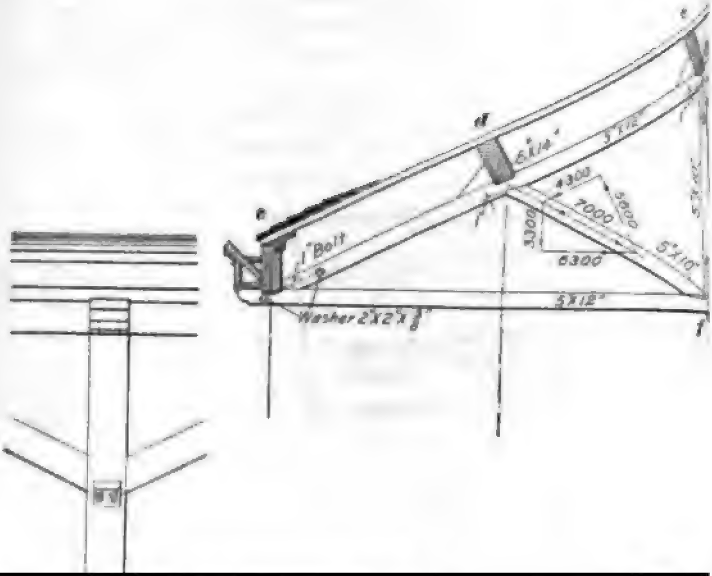
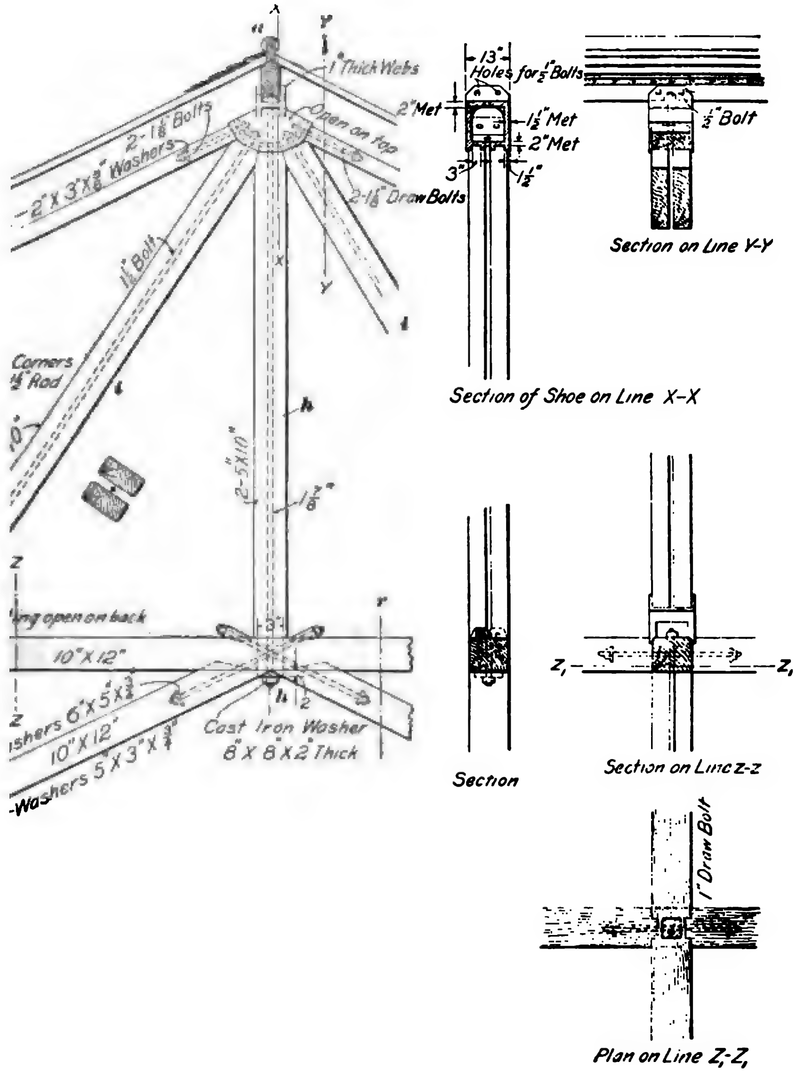


1" Bolt
Washer 2"x2"x
4300
3000
7000
3300
6300
5"x10"
5"x12"



' Hexagon Head and Nut

Iron Shoe Bedded in Cement

etal

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braces, as shown. In order to take the thrust of the knee brace on the horizontal piece, it is necessary to introduce a straining post, as at *h*, and to complete the frame by the insertion of the oblique members *i*, *i*. The cantilever portion of the frame is usually built of light timber.

In designing this structure, it must be remembered that there may be considerable action of wind from beneath the truss, which tends to reverse the stress in the several members. This reversal of stress is taken care of by draw-bolts and tension rods, as shown, and the principal junctions of the frame are made rigid by means of cast-iron boxes of suitable form to take the bearing of the members and provide for turning the nuts on the tension rods. The main posts of the frame are set in a cast-iron shoe, which is securely bedded in a cement concrete footing forming the pavement beneath the structure. In designing this shoe, an effort should be made to keep the ends of the timber dry from rain water and snow, for at this point the decay of the structure will commence. The further details of this truss are self-evident and need no explanation.

COMPOSITE TRUSSES

16. Simple Composite Trusses.—While a truss constructed entirely of timber is usually stiffer than a truss constructed partly of steel and partly of timber and known as a **composite truss**, the latter usually presents a neater appearance and possesses considerable advantage in the fact that the joints and connections are made with greater facility and additional security. The composite truss also possesses an advantage in that it does not obstruct the light nor cast a heavy shadow from the headlight or lantern, which would exist with heavy timber trusses. Besides, many of the members in the composite truss are light steel rods or tension bolts, which accumulate little dust and dirt, compared with the broad flat edge of heavy timbers. They are therefore cleaner and in every way more finished than a construction entirely of wood.

The simplest form of composite truss is a **king-post truss** provided with a tension rod of steel or wrought iron. The foot-connection of such a truss is shown in Fig. 18. This truss is reasonably cheap in construction, because there are few cuts to be made, and the saving in work on the job will more than compensate for the additional cost of the wrought-iron members.

In the construction shown, the upper chords are composed of $4'' \times 6''$ yellow-pine pieces, while the tie-member consists of a $1\frac{1}{8}$-inch wrought-iron rod, which, for greater security, might be upset on the ends. It is advisable to support this rod at the center by means of a light wrought-iron bar secured to the apex of the truss. In forming the heel

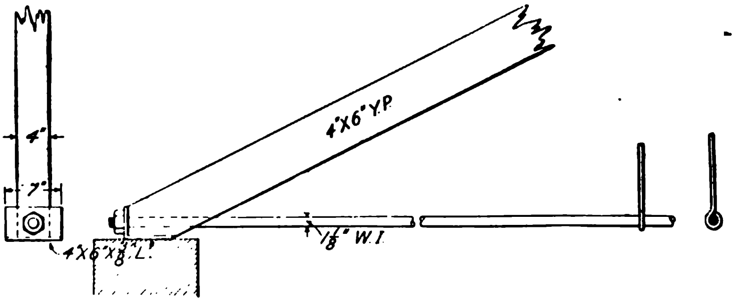


FIG. 18

connection, a $4'' \times 6'' \times \frac{3}{8}''$ angle is used, which serves the double purpose of a washer and a bearing plate. The apex of the truss is constructed by merely butting the members, preventing them from moving by dowels or by fish-plates bolted on each side. Such a truss as this may be employed for spans as great as 25 feet; but for larger spans, heavier timbers and cast-iron bearing blocks should be provided.

17. A useful type of truss for a building of about 40-foot span is shown in Fig. 19. It is difficult to classify this truss from the fact that it combines the essential features of both a king-post and a scissors truss. The lower chord member of the truss consists of $1\frac{1}{2}$-inch rods connected at the

heel to the rafter members by means of a special cast-iron washer and secured to the center suspension rod at its lower end with a 2-inch pin. In order to make this connection, it is necessary to clevis the ends of both tension rods making

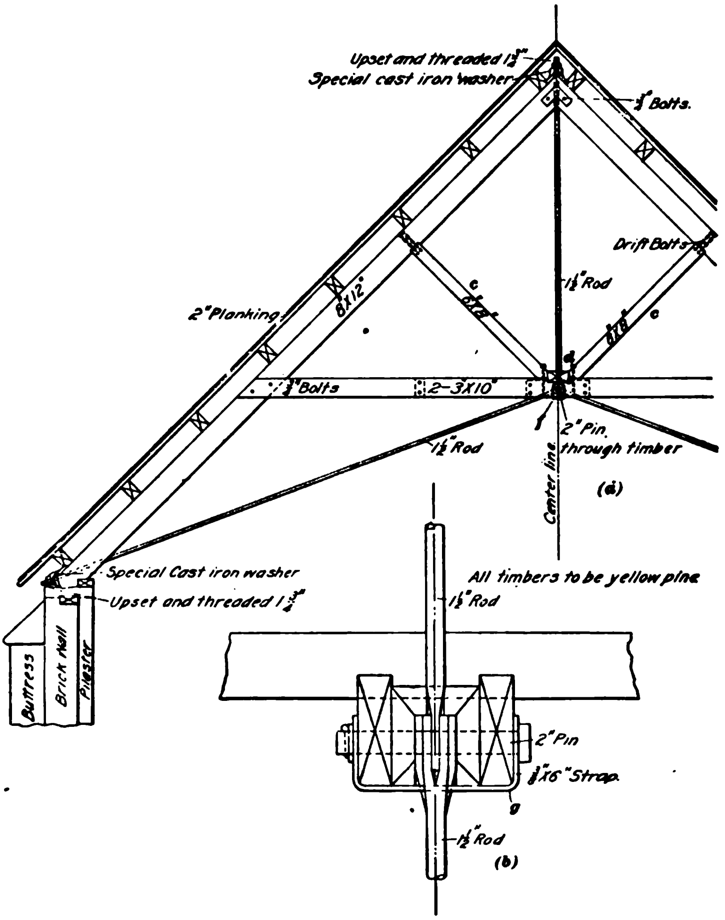


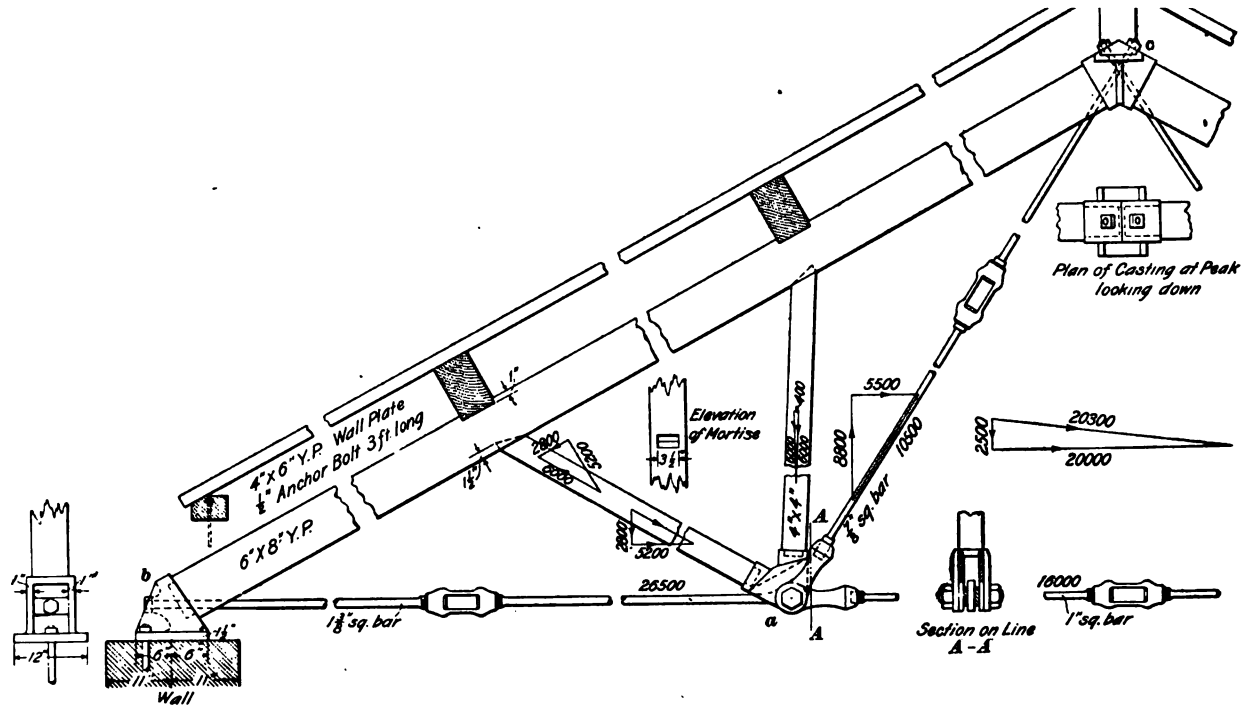
FIG. 19

up the chord, and to form an eye on the vertical suspension rod. A detail of the pin connection is shown in view (*b*). The horizontal timber members, which are composed of

two 3″ × 10″ pieces bolted to the rafter members, would, in an ordinary truss, be considered as a collar beam, and would act as a tie-member; in this truss, however, all the tensile stresses are carried by the rods, and the horizontal members form struts that brace the rafter members against deflection produced by the load of the roof covering. The struts *c*, *c* may be doweled into the rafter member or held in place by threaded bolts driven through the rafter and into the ends of the struts. The trusses may be braced laterally by a plate or sleeper, as at *d*, which tends to preserve the verticality of the truss. The joint at the apex is formed by butting the members and using on each side angle plates that are held with ¾-inch through bolts. Additional bearing for the ironwork on the timbers at the pin connection is secured by the introduction of a wrought-iron strap, which is shown in side view at *f* in (*a*) and in end view at *g* in (*b*).

18. Composite Trusses of Medium Span.—As it is ordinarily designed, the Fink truss has an odd number of panel points on its upper chord; it sometimes becomes desirable, however, to have an even number of panel points, in which case a design similar to that shown in Fig. 20 may be used. By employing the composite type of truss for this construction, an excellent appearance is obtained; this truss may be used for gymnasiums, dining halls, and other rooms or buildings of a somewhat finished character.

In the design shown, the single strut at the center is replaced by two struts that extend to the points of support for the purlins. When this construction is employed, the three tension rods meet at the joint *a*, and in order to provide adjustment throughout the truss, either turnbuckles or swivels should be used on all members. If such means of adjustment is not employed, there is a possibility of the stress on the tension rods not being equalized and more stress may be thrown on one rod than it will be able to carry. In the adjustment of the tension rods of a Fink truss of a composite type, great care should be exercised, for the rafter member is readily forced out of line and



Plan of Casting at Peak looking down
4"X6" Y.P. Wall Plate
½" Anchor Bolt 3 ft. long
6"X8" Y.P.
Elevation of Mortise
4"X4"
5500
8800
10500
⅞" sq. bar
20300
2500
20000
2800
5200
6200
26500
1⅜" sq. bar
16000
1" sq. bar
Section on Line A-A
Wall
12"
a
b
c
A

FIG. 20

an excessive camber given it, which tends to strain the timber beyond its elastic limit.

In the erection of this truss, the pieces of the triangle *abc* should be built up first and the rods properly tightened to give the rafter members some little camber. The tension rod at the center may then be drawn up so that the timbers will have a square bearing at the castings at the heel and peak of the truss. When this truss is erected in one piece, temporary collar beams, or cross-pieces, of timber should be screwed or spiked to the truss, so that there will be no tendency for the two ends of the truss to contract and thus create compression in the central tension member. A special casting provided with brackets on which the ridge pieces take their bearings is employed at the peak. These brackets are provided with lips on both sides so that the alinement of the ridge piece is secured. It will also be observed that a special casting is required at the pin connection *a* of the strut and for the bearing of the truss on the walls at *b*.

19. Composite Church Roof Trusses.—A design for a Fink truss in which three intermediate purlins are supported and which is of such a pitch as to be applicable to church roof construction, is shown in Fig. 21 (*a*). In this truss, as in the one just described, the two rods forming the truss rods of the rafter member, and the tie-rod for half of the truss, are connected at a single point by means of a pin, as at *o*; and the rafter member, besides being provided with the main truss rods, is intermediately trussed with smaller rods.

The truss shown in this figure is interesting because it is a pure type of Fink truss. Analytically, it consists of two trussed beams strengthened by 1¼-inch auxiliary truss rods. These auxiliary rods are not tightened by means of nuts at the ends, but an iron straining block is used; this consists of a special casting, shown in detail at (*b*), fitting over the end of the short strut, as at *c*. The casting is provided with a smooth hole in the center through which a straining bolt is passed. The bolt is raised or lowered by

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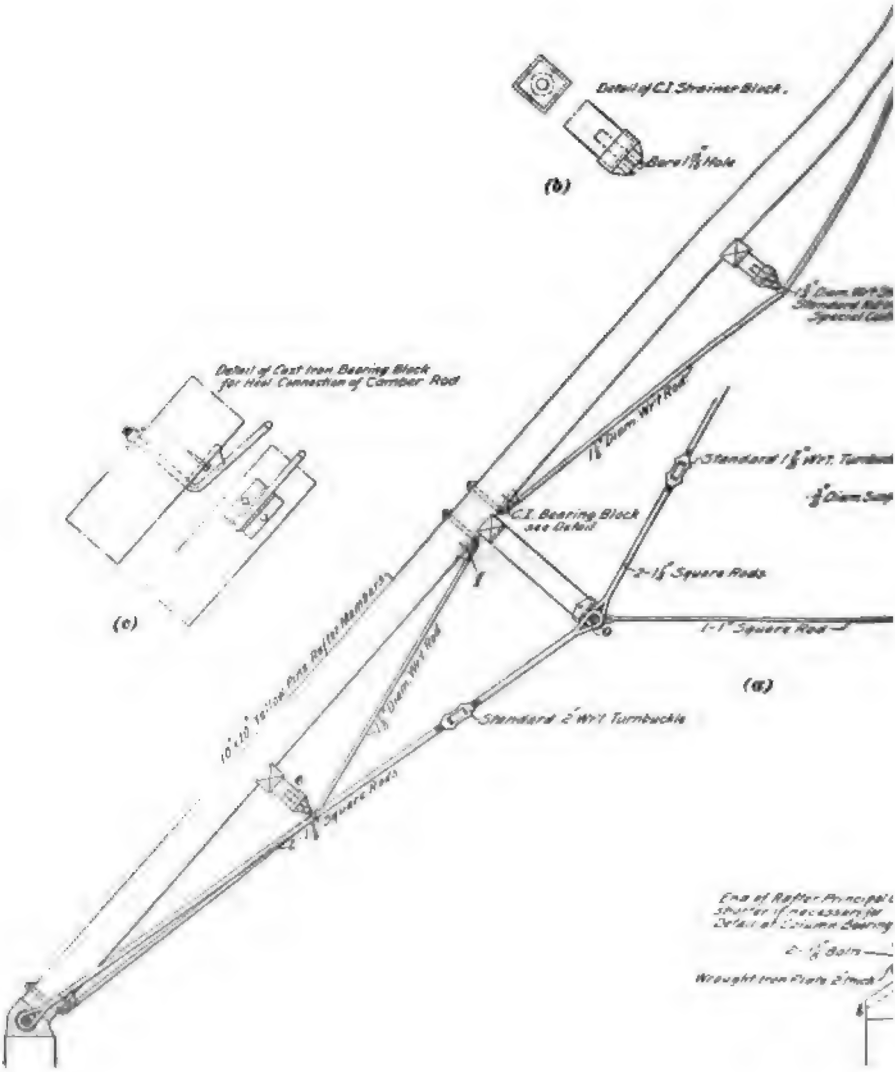
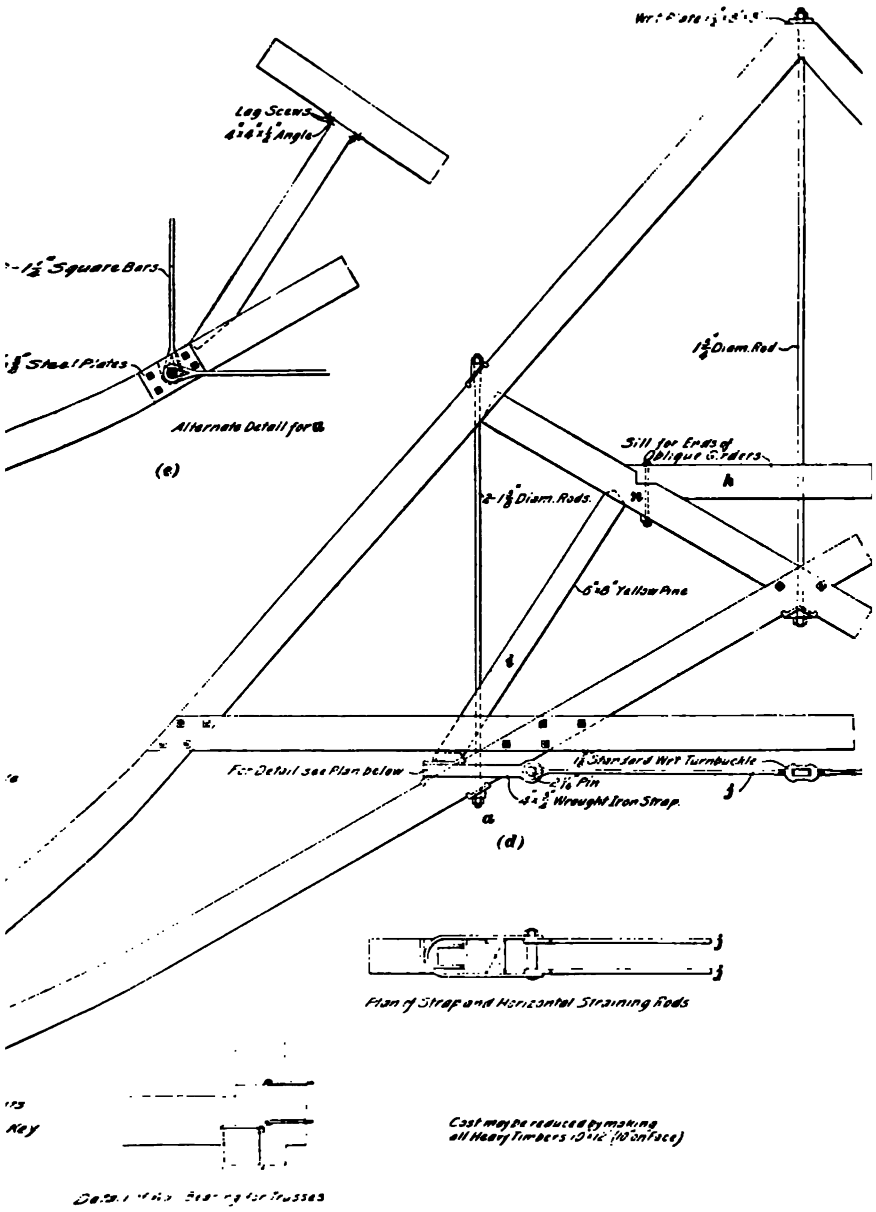


Detail of C.I. Strainer Block.
Bore 1⅛" Hole
(b)
Detail of Cast Iron Bearing Block for Heel Connection of Camber Rod
(c)
C.I. Bearing Block see Detail
2-1⅛" Square Rods
1-1" Square Rod
(a)
Standard 2" Wr't Turnbuckle
10"×10" Yellow Pine Rafter Members
Square Rods



Lag Screws
4"x4"x½" Angle
1½" Square Bars
Steel Plates
Alternate Detail for a
(e)
1¾" Diam. Rod
Sill for Ends of Oblique Girders
h
2-1⅛" Diam. Rods.
6"x8" Yellow Pine
i
For Detail see Plan below
Standard Wrt Turnbuckle
Pin
Wrought Iron Strap.
j
a
(d)
j
j
Plan of Strap and Horizontal Straining Rods
Key
Cost may be reduced by making all Heavy Timbers 10"x12" (10" on Face)

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turning its nut, which rests against the face of the casting; this movement changes the tension of the $1\frac{1}{8}$-inch truss rod, as the end of the bolt forms a seat for the latter.

These auxiliary rods are sometimes designed to pass through the timber without using the casting shown at *f*, and in detail on the drawing in (*c*). This is extremely bad practice, owing to the fact that any strain in the tension rod will tend to cause it to cut into the timber, as it has very little bearing at the edge of the hole through which it passes. This casting is let into the compression member and is an important feature of the design; without it the detail would be rendered useless. The lateral thrust of the truss is not great from the fact that the roof is very steep, and consequently the stress in the rod at the center is small, and in this instance only a 1-inch square rod is required. This rod is tightened by means of the standard turnbuckle, and should be held against sagging by means of a light suspension rod, as shown at *g*.

20. Another truss that may be used for the support of a church roof and the superimposed dome at the intersection of the nave and transept is shown in Fig. 21 (*d*). It is designed to support a heavy roof load, as well as the great weight of a timber and sheet-metal dome; this truss and the one shown in view (*a*) are suitable for spans between 50 and 55 feet. The sill *h* carries the heavy girders that form the sill plan of the dome. The reaction at the end of the timber supporting the girders is taken up by the strut *i*, and the entire truss is tied against spreading by the two heavy tension rods *j*, *j*, the strength of which is realized by means of the wrought-iron strap forged from a $\frac{3}{4}$-inch bar and the special casting, which is further explained by the plan.

This truss is one of four trusses forming a square on the plan and mitering at the point of support, as shown in the plan designated as Detail of Wall Bearing for Trusses. The thrust of the compression member at the heel of the truss is considerable, so that a special casting as at *k*, is required; the distance *l m* is such as to provide sufficient shear to

nection *a* on account of the e
special casting. While the constr
secure and perfect in every way, a
in view (*e*); the use of this detail,
ting the timber to a great exten
mended. There are three ways of
one, by mortising or notching the
heavier members, as shown in vi
angle clips and lagscrews, as sho
providing cast-iron bearing block

21. Composite Trusses Wi design of truss of the composite constructed as the Fink. In Fig. a span of 80 feet from center to c the rafters and struts are of timber of steel bars. This truss makes the interior of the building, and h reasonably cheap and entirely serv ber may be composed of a solid up of two narrow timbers bolted t the usual wooden blocks for sepa stress in the struts is not such as to be made of a solid piece of timb structed as illustrated in view (*a*) ber has a camber of 23 inches, w appearance of the truss and gives a

tie-member calculated to resist the stress caused by the lateral spreading of the two complete sections or rafter members of the truss. Additional material, however, is introduced in the portion of the lower chord extending between the heel joint and main strut in order to transmit the stress in the central tie-member to the abutments. The heel and peak connections of the truss are made with special castings, details of which are shown in views (*b*) and (*c*), that are heavily webbed and bored for the pin. The peak connection is provided with lugs, to which the ridge piece may be secured with lagscrews.

A detail of the main strut member is shown in (*a*), and the several details may be studied from the side and front view of the casting at the end connection of the strut. It will be noticed that these castings are so arranged that all the tension rods assembled at the joint may be readily connected to the pin without interference, and that the lug on the castings at *e*, *e* is made equal in thickness to the distance between the two timbers making up the strut member.

The wooden blocks forming the separators are gained slightly into the cheek of the timbers and are secured in place by means of through bolts. Through bolts are also used to secure the castings to the ends of the strut members. In order that the nuts of the pin may have a true bearing against the casting, it would be advisable to plane the surfaces at *f*, *f*, and in all good construction the holes in the castings should be bored accurately for the pin. It is necessary to mortise the rafter member at its connection with the main strut, in order to allow for the passage of the auxiliary truss rods *g*, *g*. This cutting does not weaken the member, because it occurs at the point of support and need not be more than $2\frac{1}{4}$ inches wide, or just sufficient to accommodate the end of the truss rod. The tie-member of the truss is, as usual, supported from the peak casting by means of a light $\frac{1}{2}$-inch round bar, which will pass through the turnbuckle when the truss has been drawn taut, and is secured to the peak casting by a clevis bolted to the projection on the casting.

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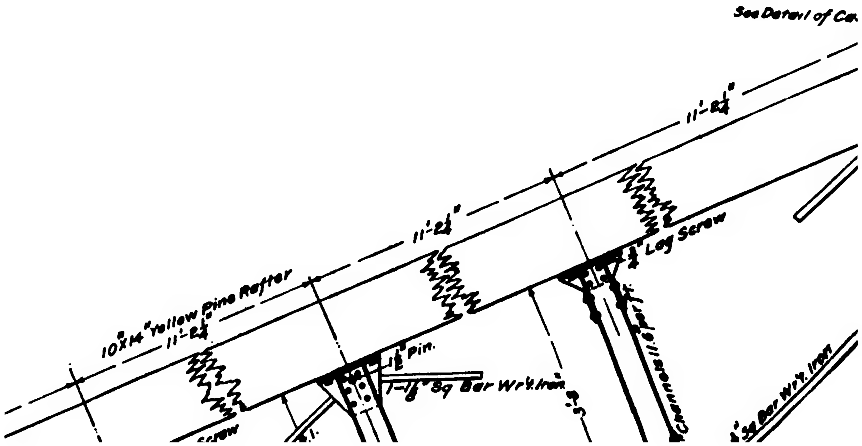
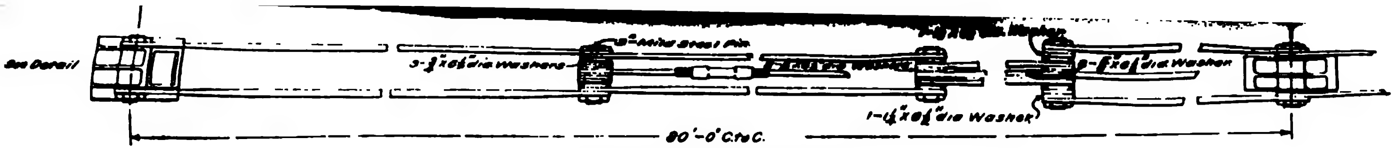


See Detail of Ca
11'-2¼"
11'-2¼"
10"x14" Yellow Pine Rafter
11'-2¼"
¾" Log Screw
1⅛" Pin.
1-1⅛" Sq Bar Wr't Iron
Channels 11.6 per ft
Sq Bar Wr't Iron



Plan of Tension Members.

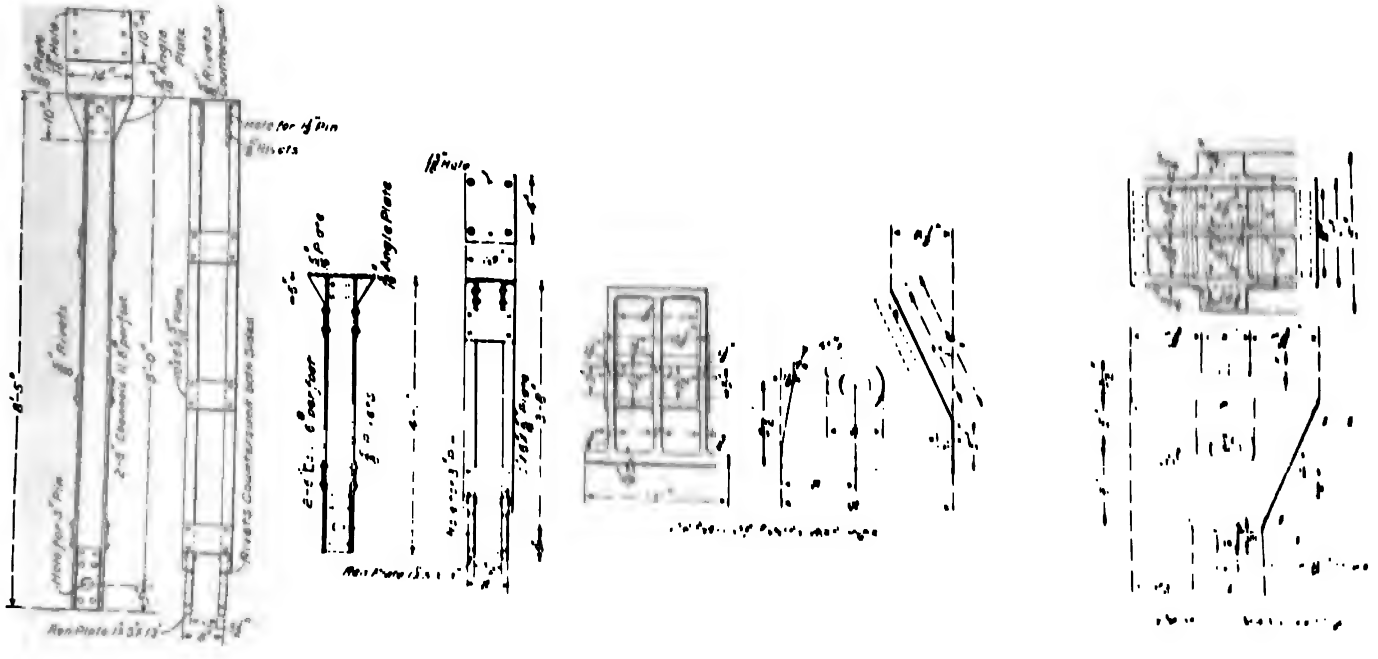


Fig. 21

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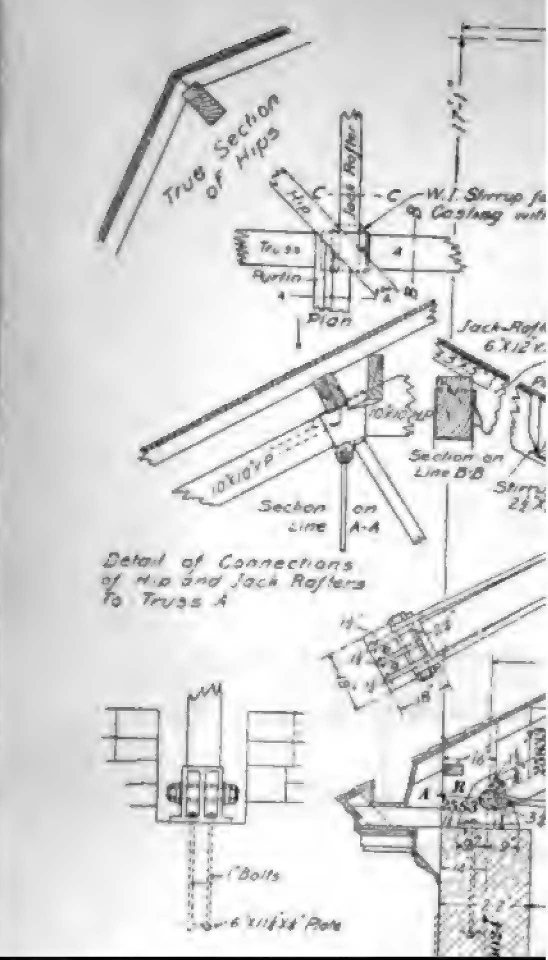
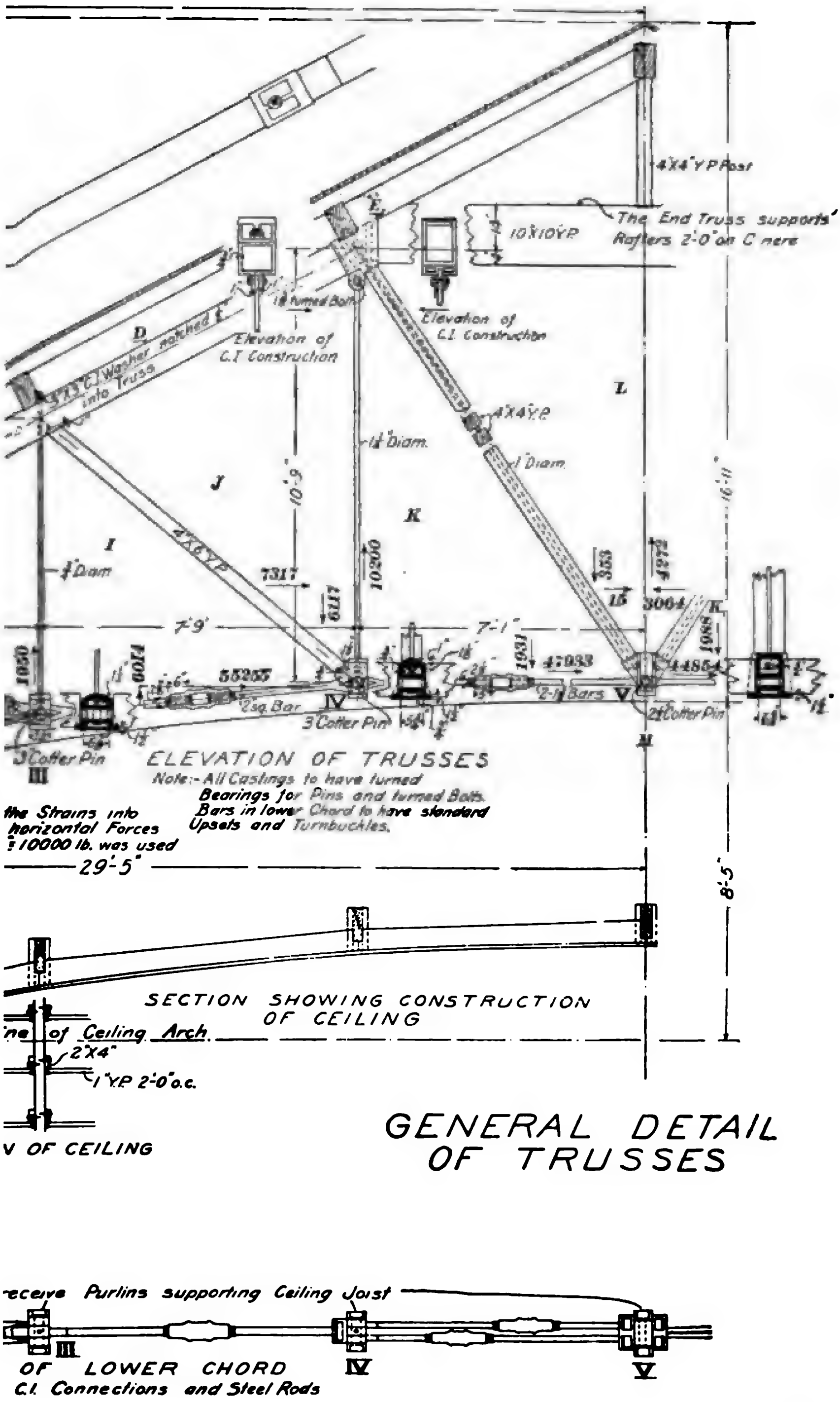

True Section of Hips
Jack Rafter
Hip
Truss
Purlin
Plan
Section on Line B-B
Section on Line A-A
Detail of Connections of Hip and Jack Rafters To Truss A
1" Bolts


4"X4" Y.P. Post
The End Truss supports Rafters 2'-0" on C. here
10"X10" Y.P.
Elevation of C.I. Construction
Elevation of C.I. Construction
C.I. Washer notched into Truss
4"X4" Y.P.
1" Diam.
4"X6" Y.P.
10'-9"
16'-11"
7'-9"
7'-1"
2 sq. Bar
3 Cotter Pin
2-1 Bars
ELEVATION OF TRUSSES
Note:- All Castings to have turned Bearings for Pins and turned Bolts. Bars in lower Chord to have standard Upsets and Turnbuckles.
the Strains into horizontal Forces
=10000 lb. was used
29'-5"
8'-5"
SECTION SHOWING CONSTRUCTION OF CEILING
of Ceiling Arch
2"X4"
1" Y.P. 2'-0" o.c.
V OF CEILING
GENERAL DETAIL OF TRUSSES
receive Purlins supporting Ceiling Joist
OF LOWER CHORD
C.I. Connections and Steel Rods

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Details of the several tension rods are illustrated in view (*d*). It will be noticed that all the rods are not provided with turnbuckles, for these are required only on the auxiliary truss rods *g*, *g* and the main tie-rod *h*, though it would be convenient to have them on the rods *i*, *i*.

22. Composite Trusses With Steel Struts.—The truss shown in Fig. 23 is similar to the one just described. The only timber member in this truss is the main rafter member, as the strut members are constructed of structural steel. The heel and peak castings of the truss are the same, in detail, as those shown in Fig. 22. The struts are of structural steel and weigh 11.6 pounds per foot. They have a greater compressive resistance than is required by the stresses in the frame, but owing to the fact that a smaller channel would cause difficulty in forming the pin connections, and inasmuch as the 6-inch channels are very light in weight, the design may be considered entirely economical. Each strut has a flat bearing on the timber of the rafter member and is secured to it by means of lagscrews. At the lower end of the channel, the flanges are cut away for the proper placing of the outside tension bars and the channels are reenforced by plates around the pinholes. It has been found, by calculation, that a 3-inch pin is sufficient to resist the stresses at the joints of the truss, with the exception of the joint at the top of the main strut member, where a smaller pin may be used. Details of the structural steel strut are shown on the drawing, and all the dimensions and explanations necessary for the clear demonstration of the design are given.

23. Composite Trusses Over Vaulted Ceiling. A carefully studied design of composite truss of special construction is shown in Fig. 24. Its peculiarity consists essentially in the direction of the line taken by the lower chord, which it was necessary to raise in this manner in order to obtain headroom to work in the coved ceiling effect that is outlined by the plaster on the drawing. The main rafter member does not extend to the peak of the truss, but

is stopped at the upper strut connection and the horizontal straining member of 10″ × 10″ yellow pine is introduced, as shown. By raising the lower tie-member in this manner, the stresses throughout the truss are greatly increased, and it is convenient to place the first strut vertical and the second and last obliquely. Owing to the fact that the truss may be unsymmetrically loaded from snow and wind, it is advisable to counterbrace the central panel by making the oblique members both tension and compression members. This is accomplished by using two pieces of 4″ × 4″ yellow-pine timber placed side by side with a 1-inch tension rod between them; this rod passes through the castings at E and M, and may be tightened by nuts at these points.

All tension members in the truss are provided with some means of adjustment, such as a nut, turnbuckle, or clevis. The castings at the joints of the lower chord member are provided with lugs on the side for the support of the ceiling joists. In the connections, pins with a turned head of small thickness on one end and a cotter pin at the other are used.

The elevation of the truss should be studied in connection with the plan of the lower chord, as the latter illustrates the necessity of sometimes using tension bars out of direct line with the stress. When the bars are arranged in this manner, the eye should be carefully considered for its bearing value on the pins, and should be so proportioned that the maximum bearing stress of the metal is not exceeded. The several details shown at the upper left-hand corner of the sheet illustrate the arrangement of the end truss with the hip and jack rafters superimposed.

STEEL TRUSSES

24. Pin-Connected Trusses of Small Span.—There is considerable demand for trusses of light construction having spans of from 25 to 35 feet. These are often built of steel to support the covering over open buildings, power houses, and boiler rooms. In Fig. 25 is shown a type

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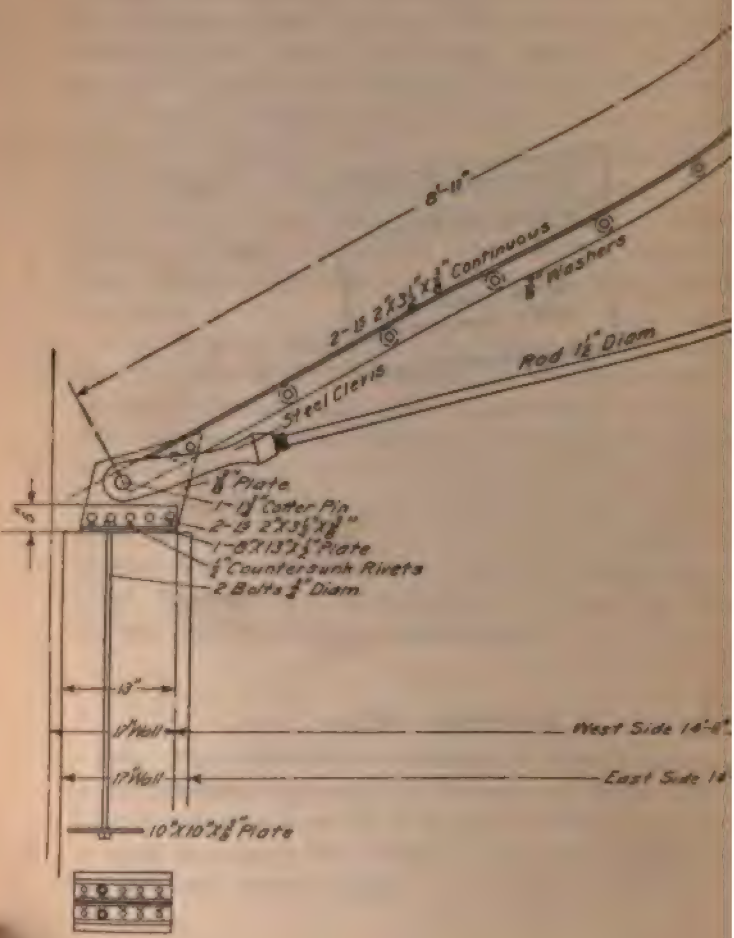
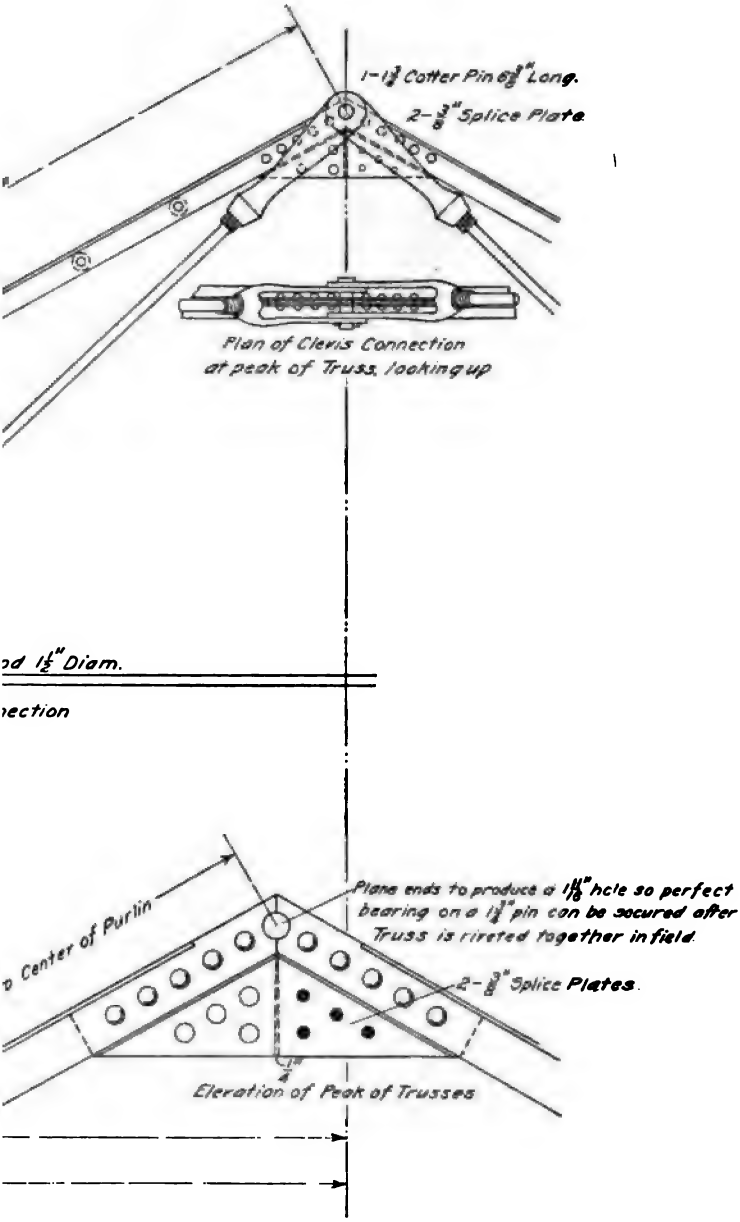


8'-11"
2-Ls 2"X3½"X⅜" Continuous
⅜" Washers
Rod 1½" Diam
Steel Clevis
⅜" Plate
1-1⅜" Cotter Pin
2-Ls 2"X3½"X⅜"
1-8"X13"X½" Plate
½" Countersunk Rivets
2 Bolts ⅝" Diam
13"
17" Wall
West Side 14'-6"
17" Wall
East Side 14
10"X10"X⅝" Plate



Plan of Clevis Connection at peak of Truss, looking up

Elevation of Peak of Trusses

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of a lightly constructed pin-connected truss with a span of approximately 30 feet. This truss consists of a top chord member composed of two $2'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles supporting a deck beam at the center and sustained by truss rods attached to the struts at *a*. The strut is built up of a plate cut to the proper shape reenforced on each side by **T**s secured in place by through rivets. The plate forming the strut is $\frac{3}{8}$ inch in thickness and the reenforcing **T**s are $2\frac{1}{2}$ inches by $\frac{1}{4}$ inch, and weigh about $3\frac{1}{2}$ pounds per foot. The rafter members of the truss are so arranged as to be sent to the field separately and are joined at the apex by means of triangular splice plates on each side, as shown in the detail.

25. Small-Span Riveted Steel Trusses.—In Fig. 26 is illustrated, in detail, a type of Fink truss having a span of 41 feet $2\frac{1}{2}$ inches. The lower chord is cambered so as to accommodate the paneled coved ceiling shown by the plaster line. The top chord consists of two $4'' \times 6'' \times \frac{9}{16}''$ angles placed back to back and separated by $\frac{3}{8}$-inch gusset plates and washers of the same thickness. The strut and tension members are built up of $3'' \times 3''$ angles, and while several of these members may have a strength greater than that required, it is usually considered good practice to use as nearly as possible the same size rolled sections throughout the truss. In view of the fact that the gusset plates were not made sufficiently large for the requisite number of rivets to be placed in the ends of the angle members, the angle-iron clips *a*, *a* are employed. These clips, besides furnishing a means for additional rivets at the ends of the members, also tend to bring the line of stress in the members coincident with the axis.

In designing this truss, it was found advisable to raise it at the end on a steel bolster, as shown at *b*. This bolster is constructed by extending the gusset plate *c* down to the bearing and reenforcing it on the bottom, sides, and top with angles, as shown. The purlins for the support of the roof consist of two intermediate 15-inch **I** beams and two 15-inch channels at the apex of the truss. These steel purlins are

connected to the gusset plates of the truss by $6'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$ angle-iron clips. In order to make a rigid connection on the lower chord at *d*, it is necessary, in this instance, to extend the gusset plate through the angles, and to use two reenforcing angles, as shown. From the location of the field rivets, it will be seen that the truss is separated into three pieces; that is, the central lower chord member and the two sections of the truss held together by it. The roof on these two trusses is constructed on 5-inch I beams for rafters, on top of which are laid the light Z bars into which are fitted book tiles, the outside roof covering being of slate, properly flashed at all points required.

26. Steel Trusses Reenforced By Architectural Treatment.—The design of truss shown in Fig. 27 is unique from the fact that it consists practically of two rafter members tied in at the feet, and securely anchored to the wall. Such construction is unusual for a truss having a span of over 50 feet, and the design would hardly be considered adequate but for the fact that the rigidity of the truss is increased by the woodwork surrounding the structural steel members. As stated in the note on the drawing, the reasons for the construction of this truss were three: The first and most important was that the steelwork was required to support the roof previous to the introduction of the finished work. Second, it was necessary to use steel members, particularly for the tie-member, from the fact that in wood they would have been unreasonably large and it would have been difficult to make the design consistent with the style of architectural treatment. Third, if the principal structural elements of the truss had been of wood, they would necessarily have been worked from the solid, and by doing this the finish of the work would have been impaired by the checking or splitting of the timber. The rafter members of the truss are composed of two 15-inch 35-pound channels secured back to back with $\frac{3}{4}$-inch separators at intervals. The tie-member consists of two $3\frac{1}{2}'' \times 2'' \times \frac{3}{8}''$ angles secured to the rafter member by means of a long gusset plate riveted between

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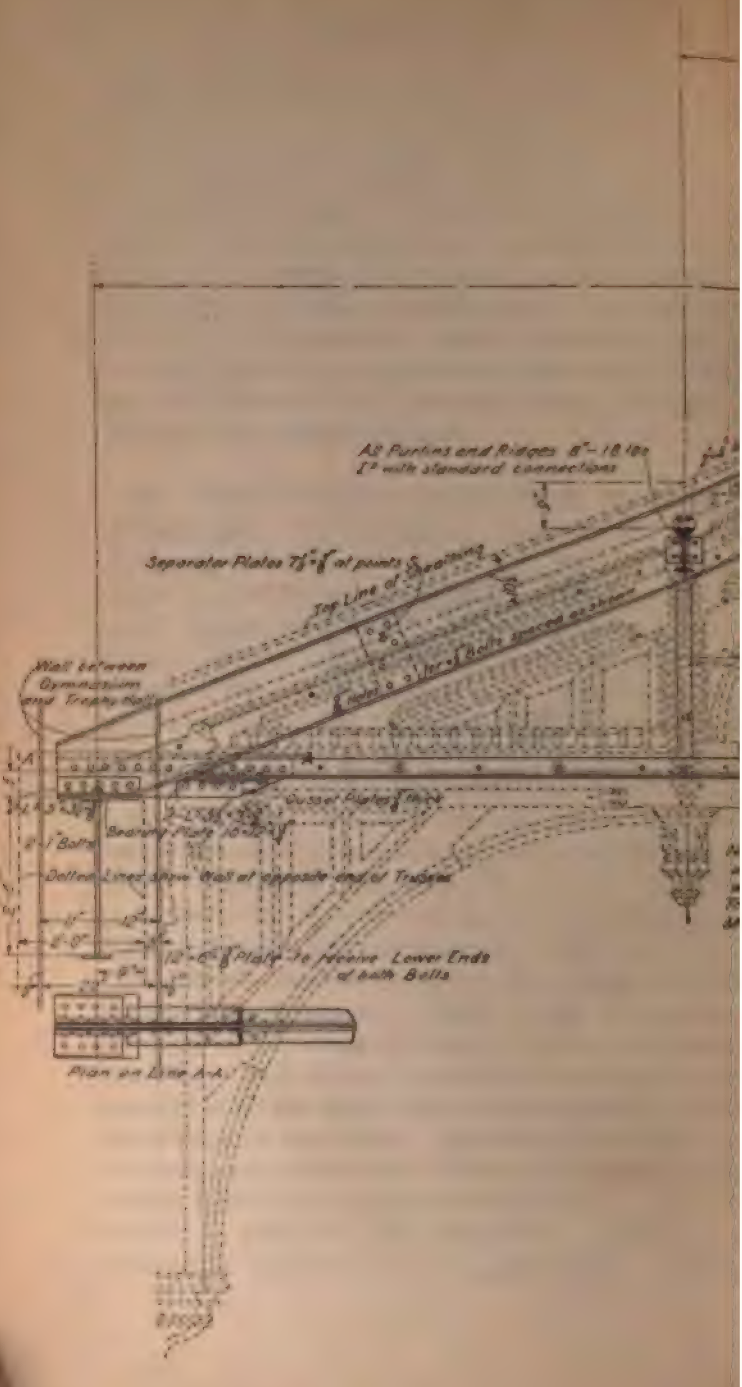
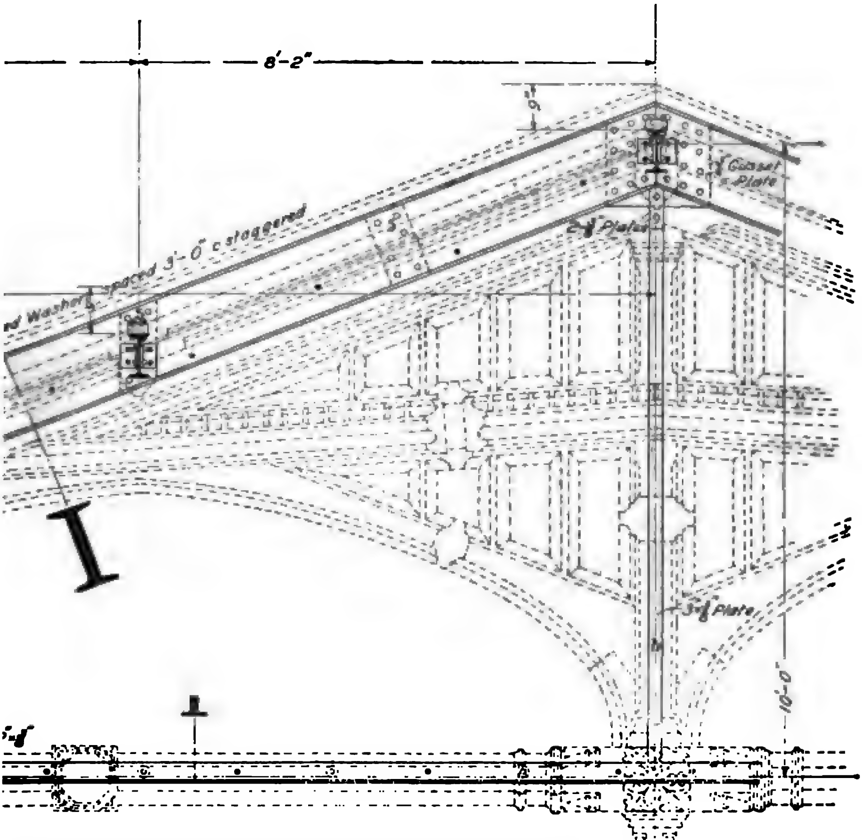

All Purlins and Ridges 8"-18 lbs
I^s with standard connections
Separator Plates
Top Line of Sheathing
Wall between
Gymnasium
and Trophy Hall
Gusset Plates
Bearing Plate
Dotted Lines show Wall at opposite end of Trusses
Plate to receive Lower Ends
of both Bolts
Plan on Line A-A



his Truss of Steel and casing it up with Wood instead of making
rate the Structural Work from the Finishing, thus facilitating the
n, protecting the Work and Workmen from the weather 2nd- If the
they would have been unreasonably large Sections. 3rd.- If the
d Sections they would be liable to be broken

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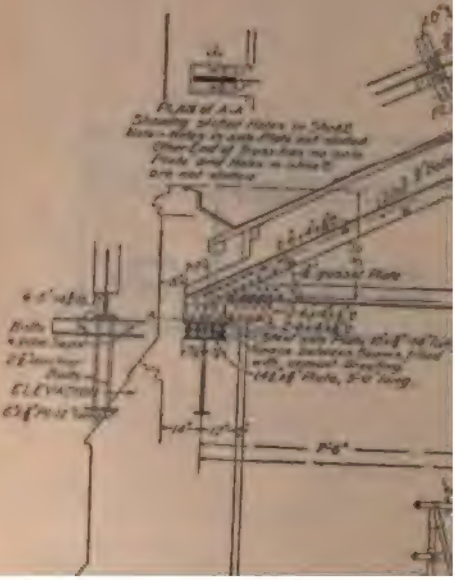
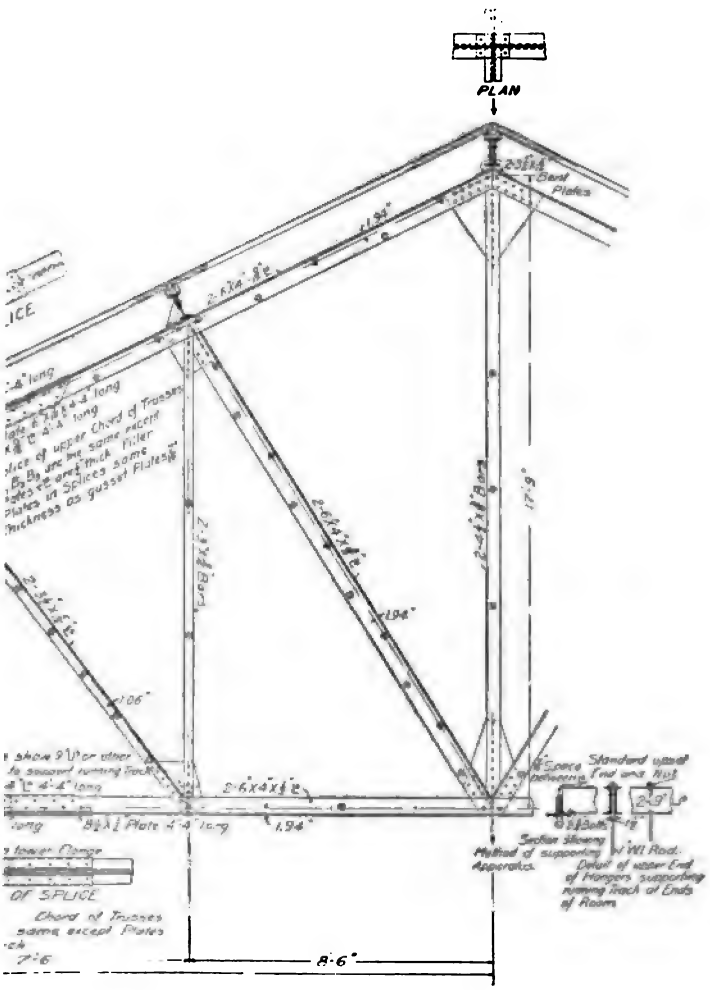

PLAN of A-A
ELEVATION
gusset Plate



PLAN
Bent Plates
1.94"
2-6X4-⅜"
2-6X4X⅜" L
1.94"
2-4⅛X⅞" Bars
17'-9"
1.06"
2-6X4X⅜" L
1.94"
8½X½ Plate 4'-4" long
OF SPLICE
Chord of Trusses same except Plates
7'-6
8'-6"
Standard upset End and Nut
Section Showing Method of supporting Apparatus
W.I. Rod.
Detail of upper End of Hangers supporting running Track at Ends of Room

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them and extending out so as to provide a fastening for the tie-member, which is secured against sagging by the 3″ × ⅜″ bar-iron plates or suspended bars, as at *a* and *b*. Since these are merely suspended bars, only one rivet is required through their ends where they are fastened to the tie-member. Throughout the lower tie-member and the rafter member, open holes are provided so that the wood may be secured to them. The purlins are of 8-inch 18-pound **I** beams provided with standard connections by which they are secured to the channel irons forming the rafter member of the truss.

27. 72-Foot Span Riveted Truss.—Trusses for large spans, when properly designed, may be constructed very lightly, as illustrated in Fig. 28, which shows a typical Howe truss constructed with angles for compression members and for the lower chord, and bars for tension members. This truss could be used for supporting the roof covering over an armory, gymnasium, or similar building; the one shown was designed for a gymnasium. It is usual in these buildings to provide a running track, which is partially suspended from the lower chord. The detail of this construction is shown in the left-hand portion of the drawing. The supports for the curb of the track, which consists of a 10-inch deck beam, are composed of two 6″ × 4″ × ⅜″ angles with a 6″ × ⅜″ filler plate. One end of these members is built into the wall, and the other is suspended from the truss by 1-inch wrought-iron rods with forged ends. Owing to the fact that the running track is curved at the end of the building, it is necessary to provide for the suspension rods by supporting, from the lower chord of the truss, **I** beams and channel irons at the several panel points along this member, as illustrated. Several details for this construction are illustrated on the drawing at the extreme right hand of the figure. The truss being of great span, it is necessary to distribute the reaction around the wall for some distance.

In order to provide a proper bearing for the truss, a bolster is constructed consisting of four 5-inch 14¾-pound **I** beams held together with bolts and separators. The trusses are

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would tend to restrict the end of the truss against movement due to changes in the temperature. All the members in this truss are composed of angles placed back to back, and in order to prevent these angles from striking one another, as well as to cause them to act together, spools or separators are inserted at frequent intervals, being placed closer together in the compression members.

From the plan of the roof it will be noticed that the two outside end trusses are securely braced laterally on account of the light construction of the end walls. At the other end, this building joins another, so that no counterbracing is necessary between the trusses *I* and *H*. All the trusses are braced longitudinally at the heel by means of horizontal and diagonal angles, as illustrated in the detail marked Section on Line *SS*. Extremely heavy splices are necessary on the lower chord of this truss, but since the upper chord is entirely in compression, only a moderately secure splicing and one that will insure against lateral deflection need be used.

Much field work is required on this truss on account of its great span and large size. The only sections that can be shipped in their entirety are the two end sections which lie between the ends of the upper and lower chords of the truss. A careful study of the details of construction and the general layout of the work, as shown in Fig. 29, is recommended, as this drawing represents the solution of a peculiar problem in conservative practice.

29. Working Drawings for Riveted Steel Trusses Having 100-Foot Span.—In Fig. 30 is shown the working drawing of the truss shown in Fig. 29 that would be made by the steel mill for the construction of the work in the shop. In general, it follows the architect's or consulting engineer's drawing, but in places the details have been changed and the sizes altered to correspond with the work of the particular shop in which the truss is constructed. The shop drawing must have every dimension and note that can possibly be required for the instruction of the workmen. It is also usual to include on the drawing the bill of

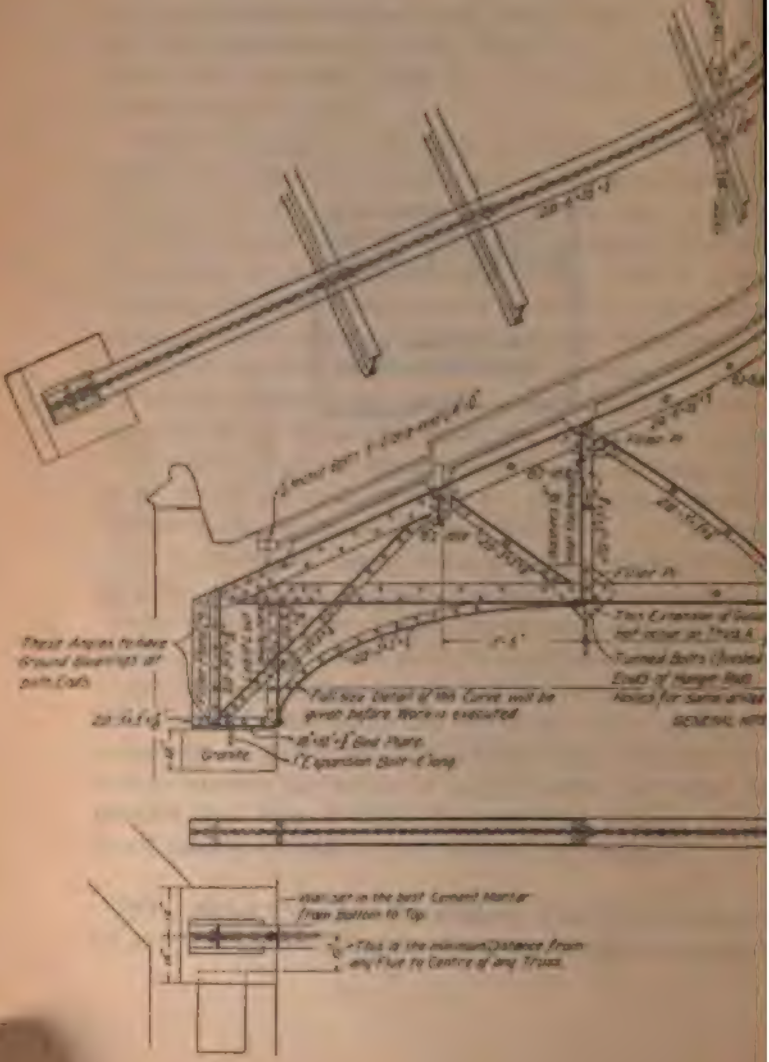
materials for the work shown; this is given in the upper left-hand corner of the figure. In the first column of the bill is designated the number of pieces required; in the second column is given a description of each piece; in the third column, the section of the required shape is denominated; while in the fourth column, in feet and inches, is given the length of the piece required. The fifth column is reserved for remarks, such as *Bent*, which would show that the plate called for was a bent plate, while *Abt* would mean that the measurement of the length was only approximate. The other remarks, such as *T-20*, are shop marks for assembling the work.

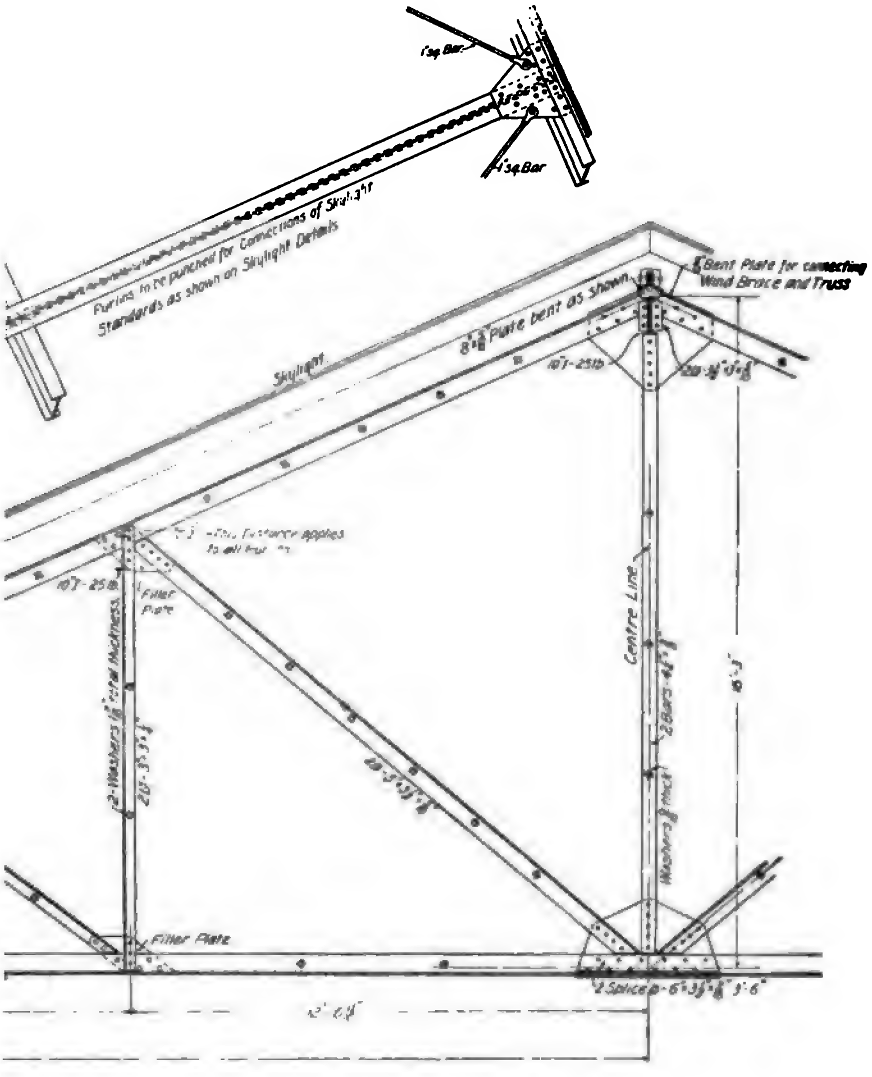
Another feature of the shop drawing is illustrated in the method by which the several angles for the different members are given. For instance, on the plan of the upper chord there are clips provided for securing lateral bracing. The angle of extension for the center lines is given by a triangle, one side being marked 12 inches and the other side being, in some cases, $6\frac{1}{16}$ inches. In all instances, it is customary to use this method of giving the angle of a member, 12 inches always being used for one side, the other measurement being determined from the drawing by calculation.

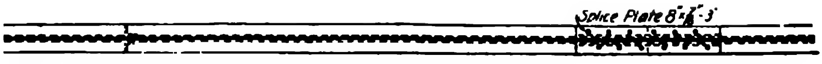
A careful study of the drawing shows that the essentials to be noted on each member are the distance between the intersections, or the theoretical joints of the structure, the sizes of the rolled shapes making up the member, with their lengths, and all the detailed measurements, such as the pitch of the rivets and the distance between separators or other details. All field rivets must be blackened in as shown, and if there are open holes or bolted connections, notes to this effect must be placed on the drawing. It is also frequently necessary to detail the connections with suitable drawings and to explain, by notes, in what manner the work shall be painted and how it shall be shipped. These details are sometimes governed by the rules of the shop.

30. Steel Trusses Peculiarly Constructed at Ends. In Art. 27, there is described a truss in which it is necessary

AND


These Angles to have Ground Bearings at both Ends
Full size Detail of this Curve will be given before Work is executed
Granite
Wall set in the best Cement Mortar from bottom to Top
This is the minimum Distance from any Flue to Centre of any Truss.





'HORD

'RUSS-B.
'EPT WHERE NOTED
'DE OF CENTRE LINE.

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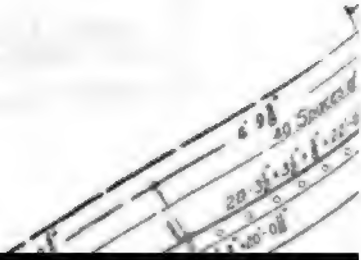

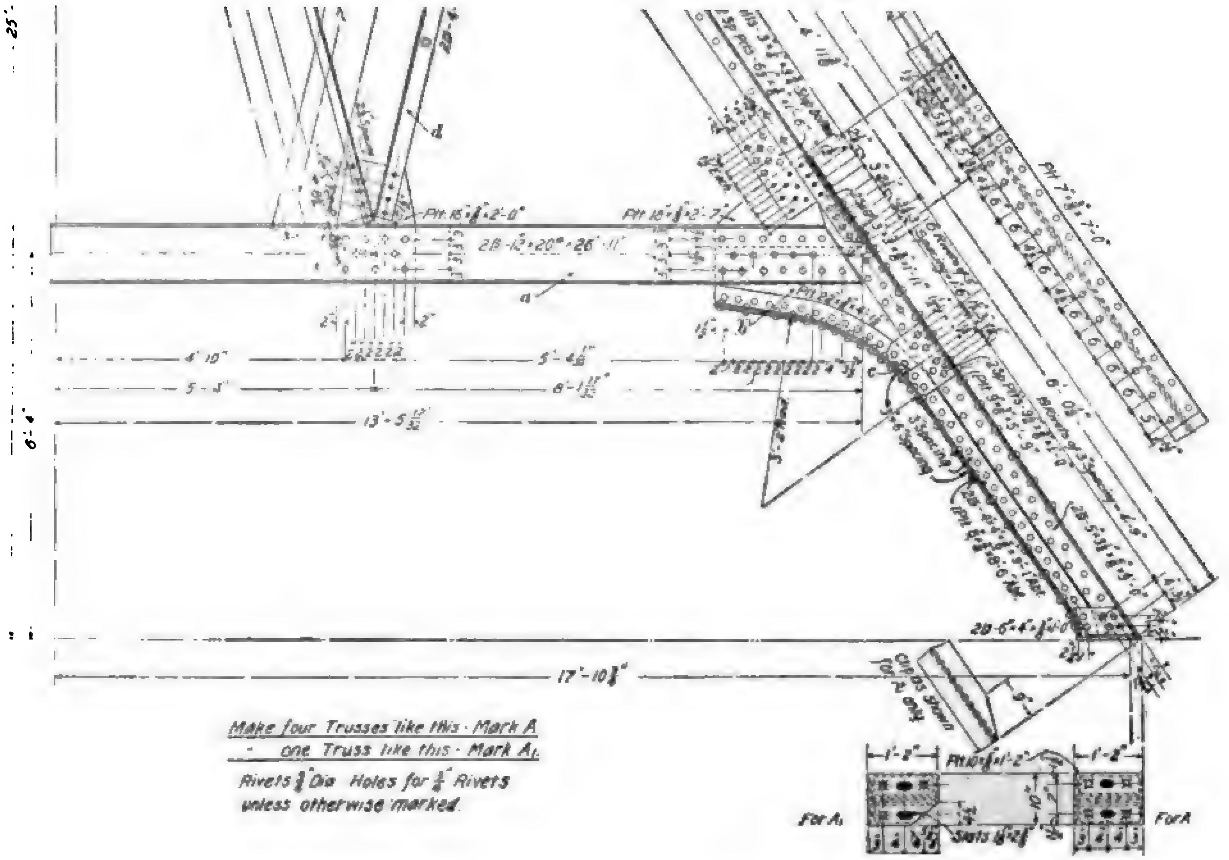
Make four Trusses like this - Mark A
one Truss like this - Mark A1
Rivets 3/4" Dia. Holes for 3/4" Rivets
unless otherwise marked.
Clips as shown for A1 only
For A1
For A
17'-10 3/4"
13'-5 17/32"
4'-10"
5'-3"
5'-4 5/8"
8'-1 11/32"
6'-0 3/8"
Pl 7"x 3/8" x 7'-0"

to raise the end by means of a steel bolster built as a portion of the truss. In Fig. 31 is shown a Howe truss having the ends similarly constructed, with the exception that it has been more carefully studied and designed. It is constructed throughout with angles placed back to back with the exception of the central tension member, which is composed of two $4\frac{1}{4}'' \times \frac{3}{8}''$ bars. The end of the rafter member of the truss is raised a distance of about 3 feet 4 inches and is held secure by a large gusset plate with a curved edge, which gives the work adjacent to the wall a good appearance. The edge of the gusset plate is reenforced by two $3'' \times 3'' \times \frac{1}{4}''$ angles, and all the reaction is taken up by the vertical angles, though the heel of the truss is reenforced additionally with oblique $3'' \times 3'' \times \frac{1}{4}''$ angles. A note on the drawing stipulates that the bearing angle at the heel shall be ground at each end of the truss. By doing this, these angles take a positive bearing and transmit the stresses to the abutments without dependence on the rivets.

31. Riveted Steel A Truss.—In Fig. 32 is shown an unusual type of truss, but one that is convenient and frequently useful. It is usually designated as an **A truss,** from the fact that it resembles that letter, and is generally used on buildings of Gothic design. The peculiarity of the truss is that the horizontal tie-member *a*, instead of being located at the heel of the truss, is placed 6 or 8 feet above that point. This construction is necessary on account of the height of the vaulted ceiling over the top floor of the building. It can readily be seen that in this truss great transverse resistance is required in the leg *b*. This portion of the truss is usually constructed of heavy angles and flange plates. Not only must the tie-member *a* supply the necessary tensile resistance, but it is also frequently required to support an attic floor, and, therefore, must be figured for transverse strength as well.

In examining the construction of the leg *b*, it will be noticed that it is composed of a web-plate, two $5'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$

outside angles, and two $4'' \times 4'' \times \frac{3}{8}''$ inside angles. The outside angles are double-riveted to the web, while the inside angles have a single row of rivets. From the fact that it would be a considerable waste of material to cut from a single plate, this member is spliced at *c*, the double splice plate being sufficient to supply the transverse resistance at this point. The tension, or inner, flange of the leg is reenforced with a cover-plate 8 inches wide and $\frac{5}{8}$ inch thick, while the outer, or compression, flange is partially reenforced with a cover-plate 9 inches by $\frac{3}{4}$ inch.

Owing to the fact that the rafter member of the truss supports a purlin at the center between the intermediate support and the tie-member, it is subjected to considerable transverse stress, particularly from wind loads, and is therefore constructed to resist this stress by using a $10'' \times \frac{3}{8}''$ web-plate and two $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles. The strut member *d* sustains considerable stress, and is therefore made of $4'' \times 4'' \times \frac{3}{8}''$ angles, securely tied together with separators at intervals, as shown on the drawing. The member *e* is a tension member and is constructed of light angles, separators being required to prevent a chattering of the rolled shapes by their knocking against each other. Owing to the shape of this truss, it was difficult to ship it, and much field work was required in its erection. The leg and horizontal tie-member were shipped in one piece. One rafter member was shipped connected with the top gusset plate, while the other was separate. The tie-member *e* and strut member were shipped separately. As one of these trusses occupied a position adjacent to two transverse wings of the building, that is, a position that in a church would be a transept, it was necessary to clip the corners of the foot-plates, as designated at the bottom of the figure. The purlins were simply connected on the rafter members by means of angle-iron clips provided with rivet and bolt holes, as designated on the drawing.

32. A similar type of roof truss to the one described in Art. **31** is shown in Fig. 33. This truss, however, has

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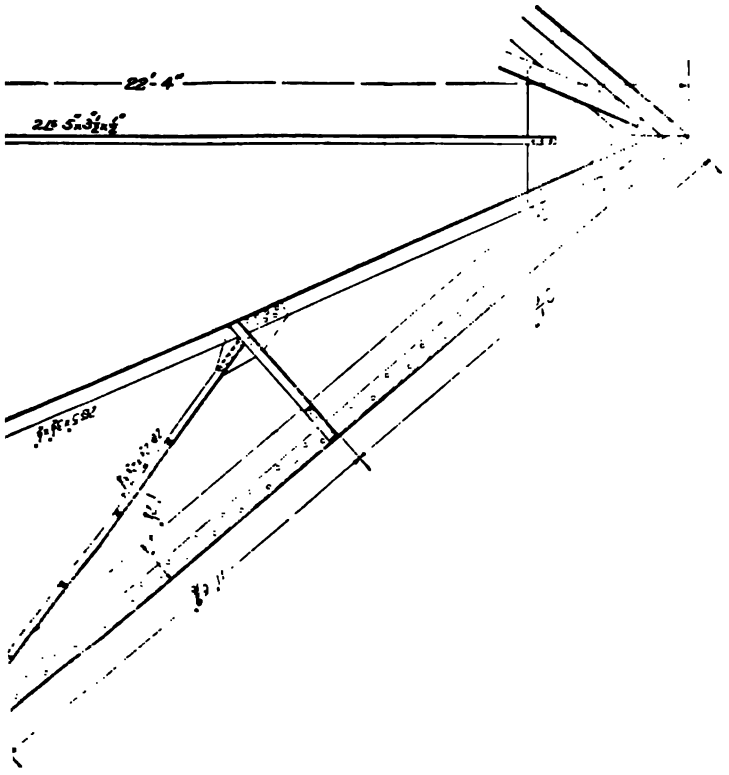
22' 4"
2L 5 x 3½ x ⅝"

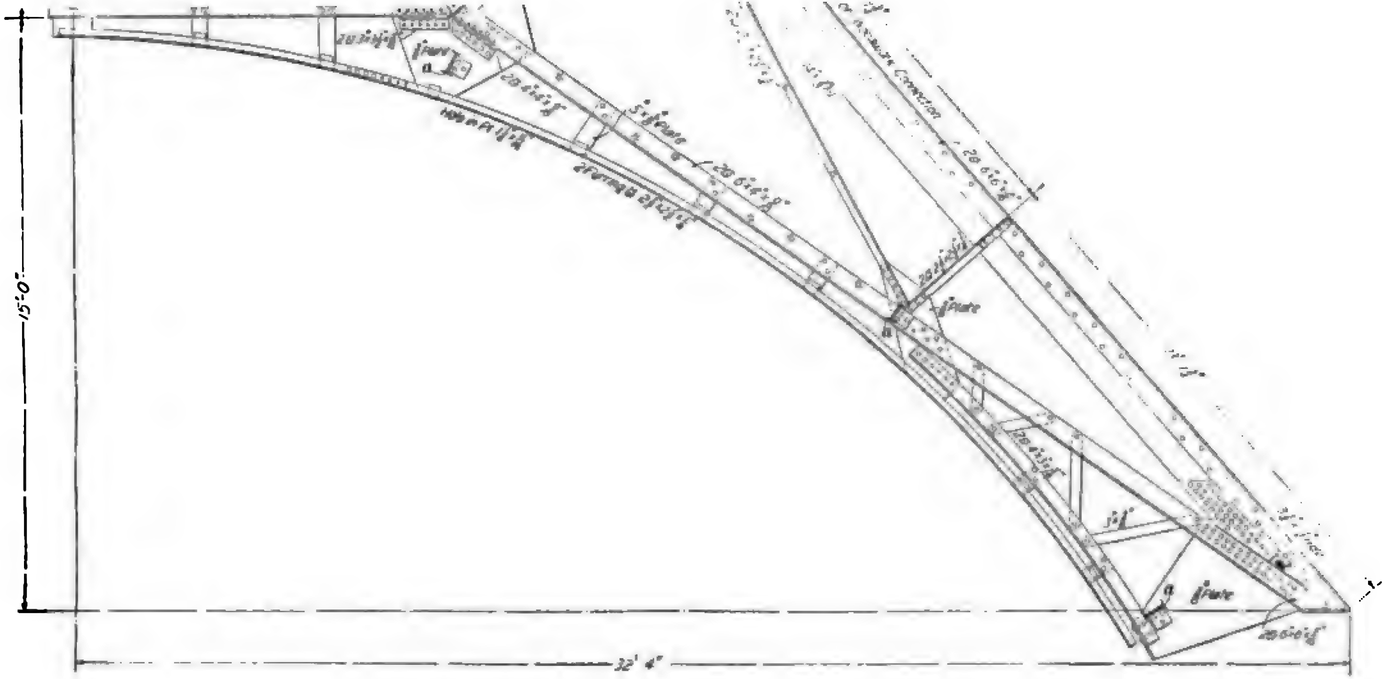

Fig. 33

§ 29

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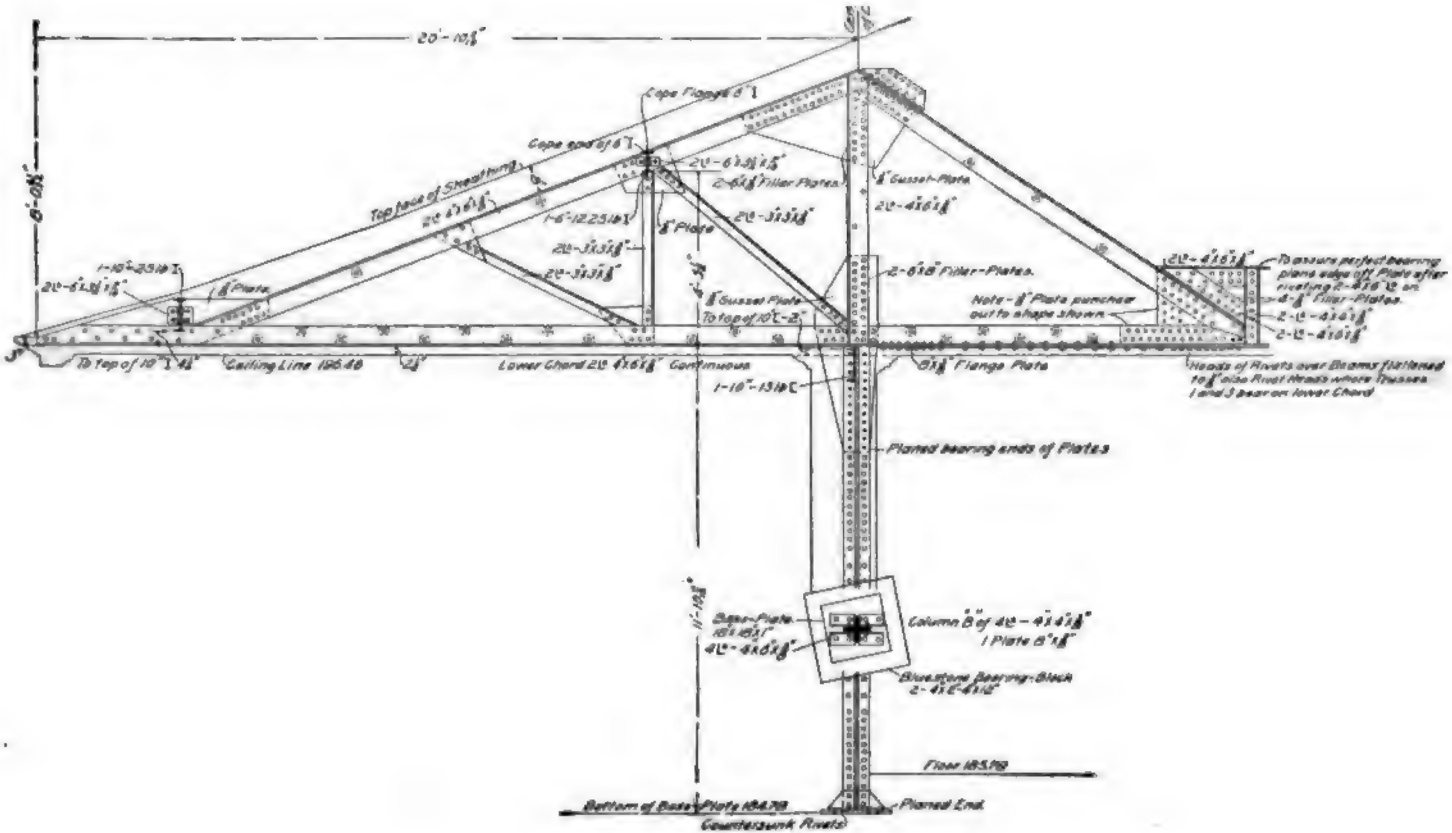

Top face of Sheathing
Ceiling Line 196.48
Continuous
Planed bearing ends of Plates
Bluestone Bearing-Block
Planed End
Countersunk Rivets

FIG. 34

a far greater span, as the span of the former is about 36 feet while the span of this one is approximately 64 feet. This truss is intended to support the roof over a vaulted ceiling, and is constructed along the lines of a Fink truss. This differs somewhat from the truss just described from the fact that there are no members subjected to transverse stress, but the horizontal tie-member is raised so high that the form of the truss complies with the requirements of a vaulted or curved ceiling. The top flange of the rafter member of the truss is provided with $\frac{9}{16}$-inch holes for the fastening of the wooden sleepers on which the purlins or roof joists may be placed and spiked. The vaulted ceiling is carried on light steelwork suspended with straps from the lower chord member of the truss, the form of the ceiling being given by two $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ angles bent to shape. This truss is designed to rest on steel columns, and it is evident that a cove must be worked in the corner of the room or hall to meet the curved ceiling supported by the truss. Small **I** beams are supported at *a*, *a* to carry the curved ceiling between trusses, the curved angles being suspended by clips and clamps from these beams.

33. Riveted Steel Cantilever Trusses.—In Fig. 34 is shown a cantilever truss constructed of steel and supported on steel columns composed of four $4'' \times 4'' \times \frac{3}{8}''$ angles and reenforced with one $8'' \times \frac{3}{8}''$ plate. These steel columns are built inside of brick piers, and the overhanging end of the truss is retained against tilting by anchoring the interior end to heavy **I**-beam lintels by means of stirrups composed of four $1\frac{3}{4}$-inch steel bolts. The upper chord of the truss is built of two $4'' \times 6'' \times \frac{3}{8}''$ angles, and the lower chord is constructed of the same sized sections.

It will be noticed in the overhanging cantilever end of the truss that the lower chord is built as a compression member and the upper chord as a tension member; this is evident from the fact that the separators are placed closer together and are more numerous in the lower chord than in the upper. The stresses in these chords will be changed by the action

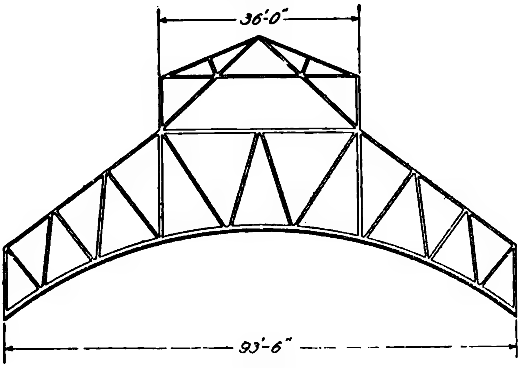

Fig. 35

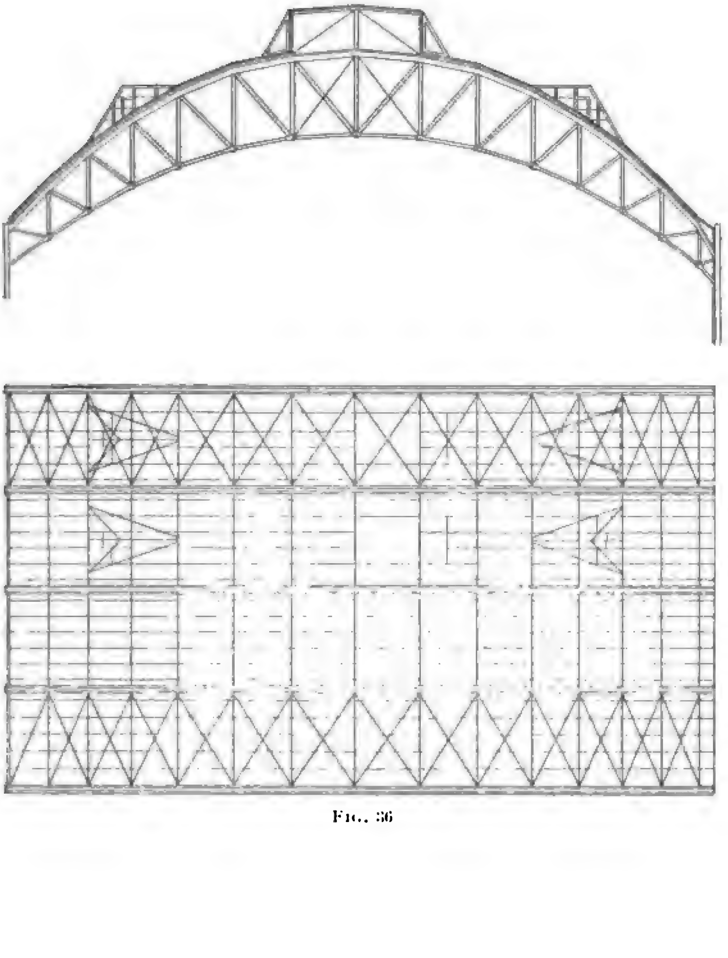

Fig. 36

of the wind beneath the truss, a condition that is quite likely to occur in open work of this character.

This truss is the hip truss for the passenger shed of a railroad station. The 10-inch 15-pound channel shown at the top of the column extends between the trusses and turns

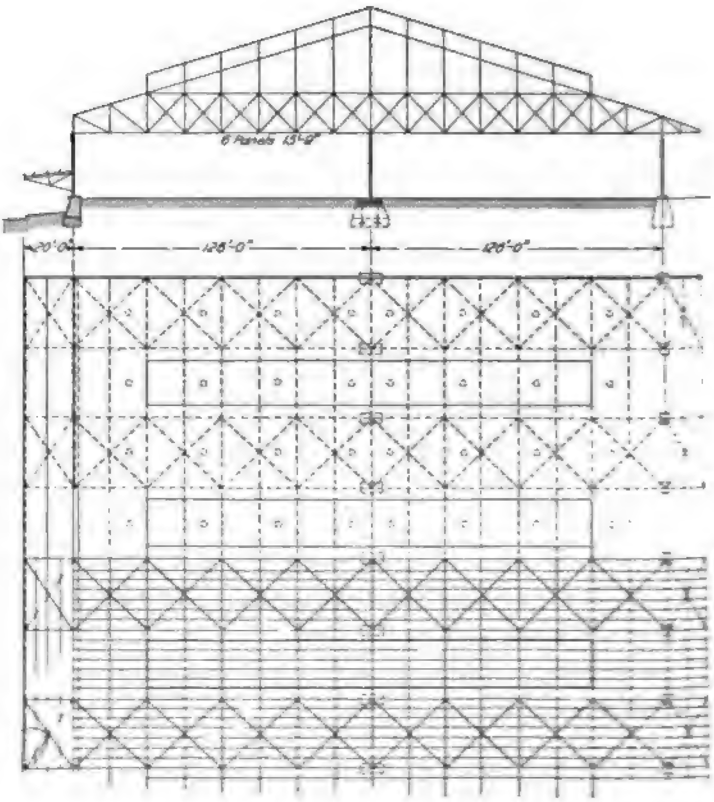

FIG. 37

the corner of the building on a curve. It acts as a lateral tie between the tops of the brick piers in which the columns are built.

34. Trusses of Large Span.—In buildings of great width, such as those used for docks and train sheds, various

of trusses are employed. In
35 is shown a truss having a
of about 94 feet, with a curved
· chord. Such a truss may be
›rted by columns, or a pair of
ɜs may abut on an arcade or
ion wall. This type of truss
good appearance and could be
for supporting the roofs over
ɜrs, auditoriums, or large as-
ly halls.

Fig. 36 is shown the elevation
plan of a somewhat similar
Dormers are constructed at
'als between the trusses and it
ırts at the crown a lantern or
ator. It will be noticed, in the
that the end trusses are wind-
d laterally in pairs, and from the
:ion it will be seen that the truss
ported on columns or posts.

In Fig. 37 is illustrated, by
ınd elevation, a type of truss
ding over three supports and
led with a cantilever end, which
ɔe used for train sheds. As
ı in the plan, these trusses are
lly braced in pairs by diagonal
rods. The span between sup-
is 126 feet, the total width of
ıilding being 252 feet. When
ıd truss is exposed, or unpro-
by a wall, it should be heavily
l laterally in order to prevent
nd from blowing it in. This
is usually constructed with
d joints.

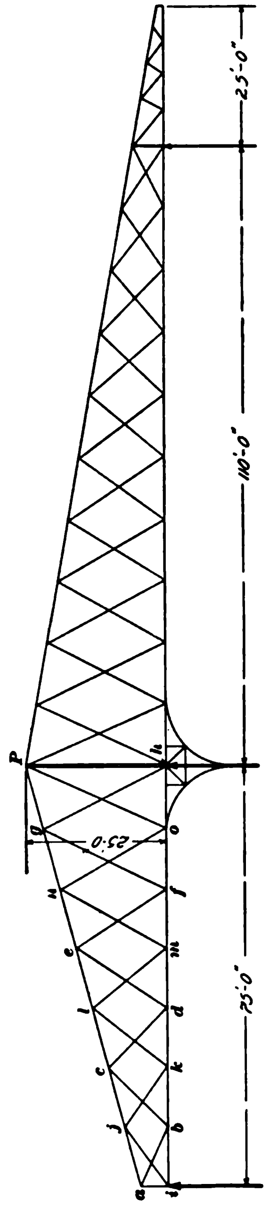

FIG. 38

An alternate design for the truss just explained, but one which does not make as good an appearance, is shown in Fig. 38. If necessary, both ends of this truss could be made cantilever, in which case the construction would be extremely heavy.

A truss that could be constructed either of steel or timber is shown in Fig. 39. Such a truss would have four columns in the width of the building, and the two outside bays would run from 50 to 100 feet in width, while the inside bay would have a span of about 200 feet. Such a truss carefully

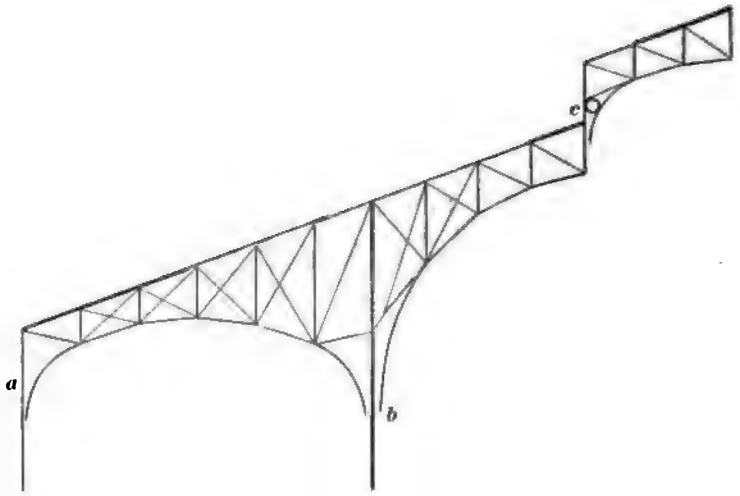

FIG. 39

designed would present a graceful appearance on the inside, and would have the advantage of a clear span and great headroom, making it very suitable for exhibition buildings.

36. Lateral Wind Bracing.—In Fig. 40 is given an example of lateral wind bracing, the bracing lying parallel with the roof slope, and consisting of square bars with upset ends provided with clevises and turnbuckles. The rods of the wind bracing are secured to the trusses by means of gusset plates riveted to the top chord. The wind bracing takes the form of a truss, and in this way provides

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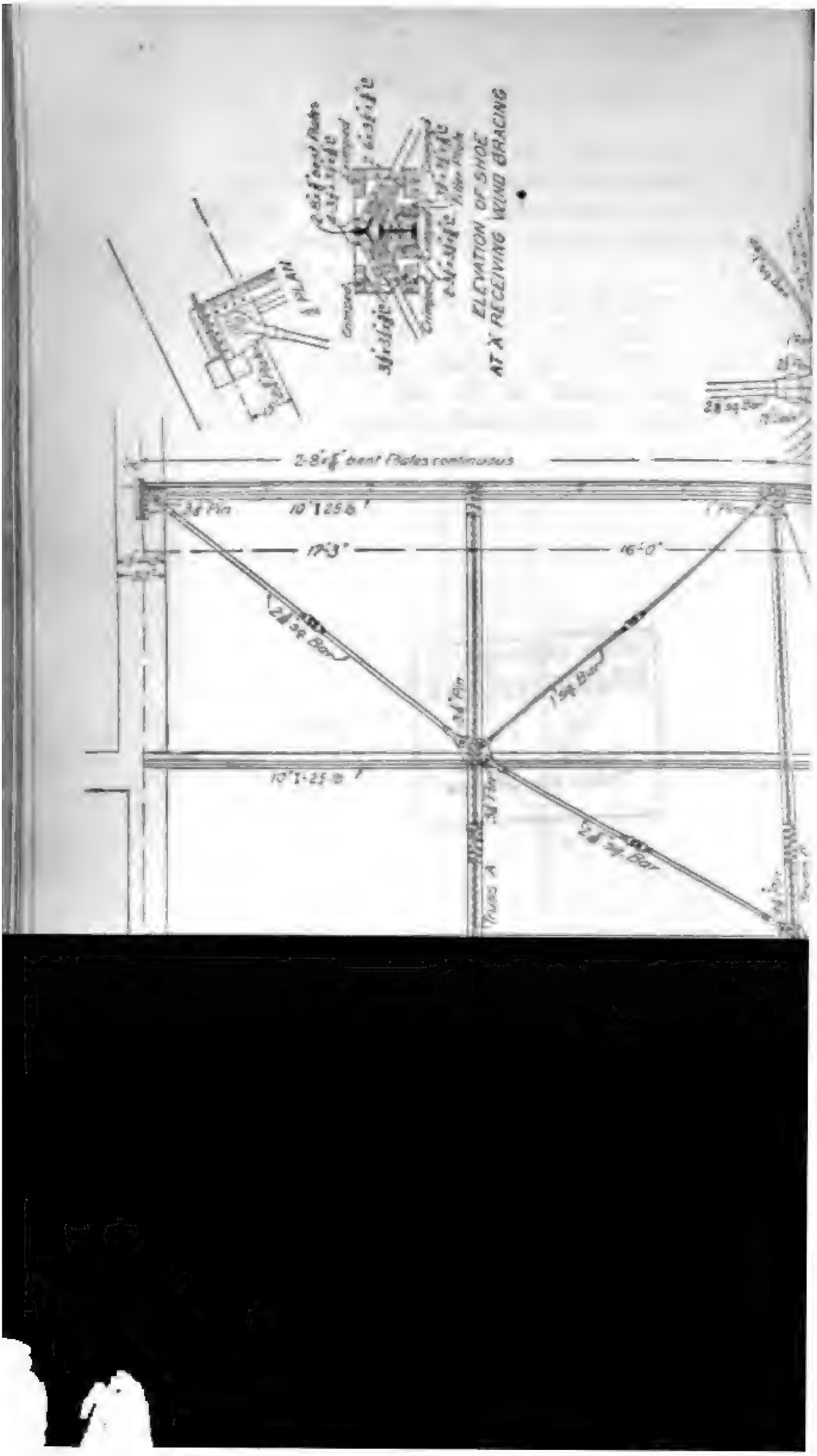
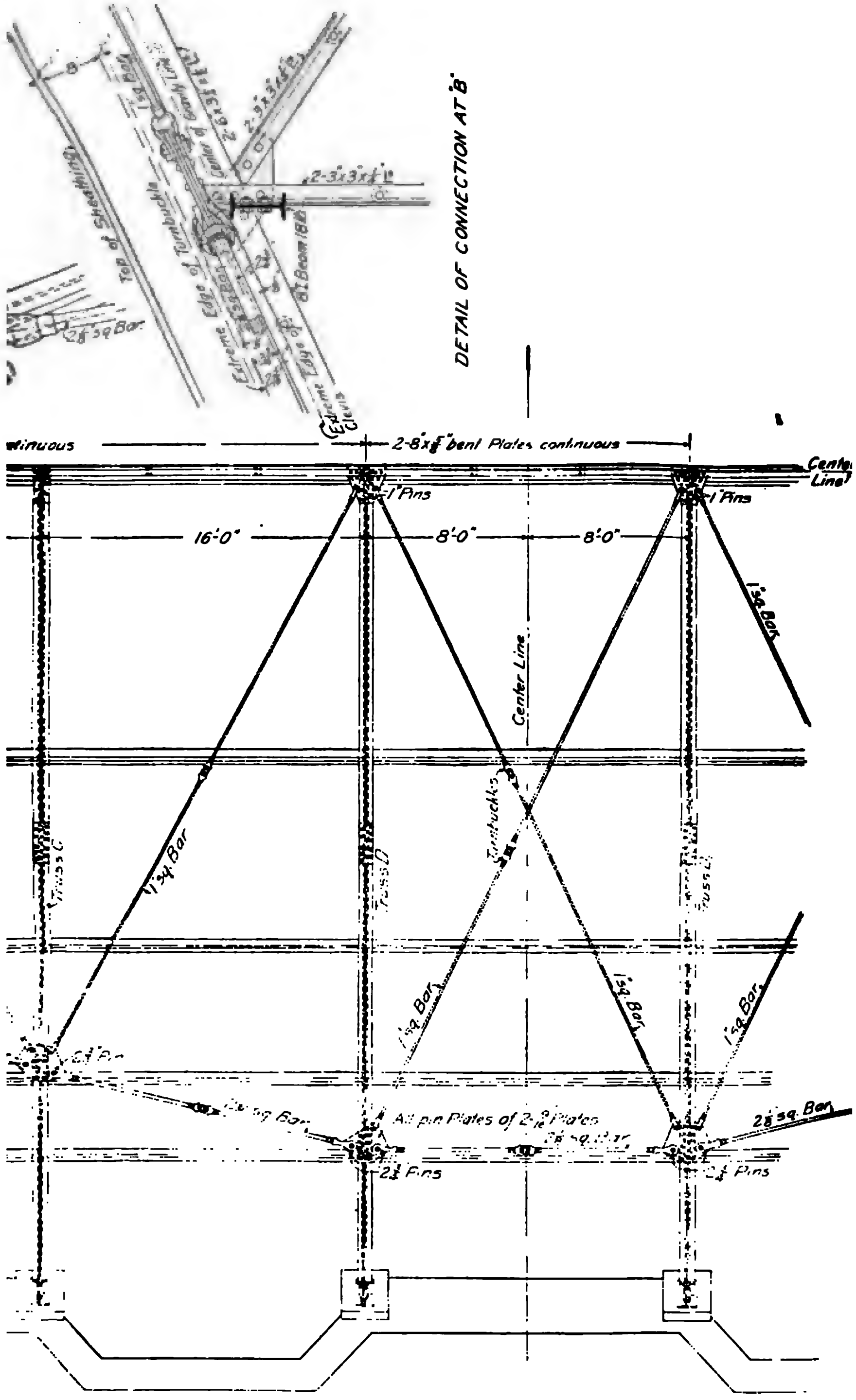


ELEVATION OF SHOE
AT X RECEIVING WIND BRACING
PLAN
bent Plates continuous
17'-3"
16'-0"
Sq. Bar
Truss A



DETAIL OF CONNECTION AT 'B'
Top of Sheathing
Center of Beam Line
Edge of Turnbuckle
Extreme Edge of
Edge of
Beam 18 B
Extreme Clevis
2-3½x3x¼
2⅝ sq. Bar
2-8"x⅝" bent Plates continuous
Center Line
1" Pins
16'-0"
8'-0"
8'-0"
Center Line
1 sq. Bar
Turnbuckle
Truss C
Truss D
Truss E
1 sq. Bar
2⅛ sq. Bar
All pin Plates of 2-½ Plates
2¼ Pins

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NDATIONS

additional lateral resistance to the wind. It is frequently necessary, where the end of the building is open and the truss is sheathed or covered, and thus a great area exposed to the wind, to provide, on the lower chord of the truss, a horizontal or lateral truss. This truss is usually suspended or otherwise supported from the truss which it braces and the adjacent truss. The details in Fig. 40 are interesting and should be carefully noted by the student.

WIND BRACING

INTRODUCTION

1. In all buildings of a greater height than 100 feet and having a width less than one-fourth the distance from the top of the foundation to the capping on the parapet, some means of lateral bracing against wind pressure must be provided. In the usual construction, the only lateral resistance exists in the rigidity of the beam connections to the columns, and in the stability of the masonry wall, as buildings generally fall within the limitations given. The conditions, therefore, that require a special study of wind bracing are practically found only in high-building construction of the **skeleton type**; that is, buildings in which the steel columns, beams, and girders form the frame, while the walls have no structural significance, but are merely curtains or screens. Such buildings are many stories in height and are often extremely narrow, but they offer a great area to the wind on the sides.

Experience has proved that skeleton buildings more than twelve stories in height, constructed with the usual cast-iron column and steel-beam connections, do not offer sufficient rigidity against the pressure of the wind on their sides. In one instance, particularly, the framework of a ten-story building of this construction, when partially erected, was found, on examination after a heavy storm, to be strained to such an extent as to be 10 inches out of plumb. This example and others that have occurred show the inadequacy of the usual connection between the columns and floor

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systems for buildings of great height. Where structural steel columns are used and the floorbeams and girders are securely riveted to them, similar to the design shown in Fig. 1, the connections are more rigid, and buildings may be constructed in this manner to the height of ten or twelve stories with no other wind bracing than the rigidity of the connections.

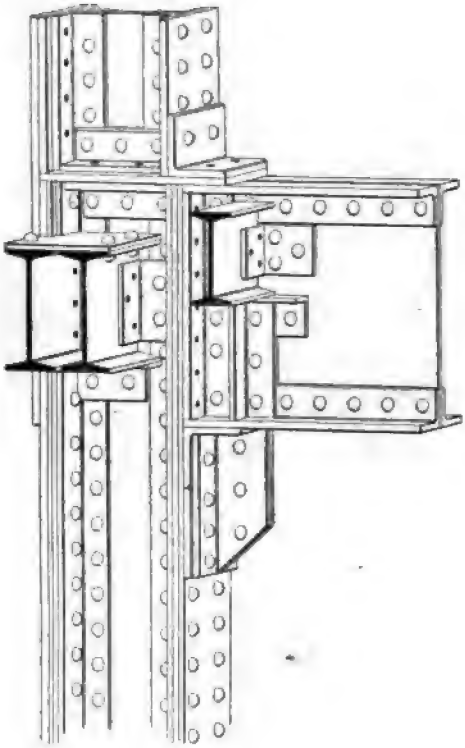


FIG. 1

2. In designing the wind bracing for buildings of the skeleton-construction type, it is usual to assume a wind pressure of 30 pounds to the square foot, because the maximum wind pressure of 40 pounds to the square foot is seldom realized on a large surface. The pressure of 30 pounds will probably never be experienced except once or twice in the life of the average building, therefore a very low factor of safety is usually adopted in the design of the wind bracing.

The rigidity of a building against lateral movement, provided by the partition walls, especially those of brick, and by heavy fireproof floors common in this type of building, is not considered in the design, and thus increases the safety. While these elements assist in resisting wind pressure, their efficiency is inadequate and they cannot enter into the

calculation for the design of the bracing. The only way in which they can be considered is to regard them as assisting the bracing to secure rigidity, and consequently to adopt a low factor of safety, usually 3, in the design of the building.

TYPES OF WIND BRACING

3. The several types of **wind bracing** that have been designed to meet the conditions in high-building construction are the *gusset-plate*, or *knee, bracing;* the *sway-rod*, or *diagonal, bracing;* and *portal bracing.*

4. Gusset-Plate and Knee Bracing. These types of bracing were designed in order to secure a more rigid connection between the floorbeams or girders and the columns. Rigidity in the usual connection is obtained, as shown in Fig. 2, by bolting or riveting the bottom flange of the beam to the supporting bracket, which is securely riveted to the column, and by likewise securing the top flange to the column, either by angle-iron clips riveted to the column or secured by bolts through the column. It is evident that in this connection any lateral movement, that is, any tendency to distort the frame formed by the column and the floor principals, will shear the rivets or bolts holding the lower and upper flanges of the girder to

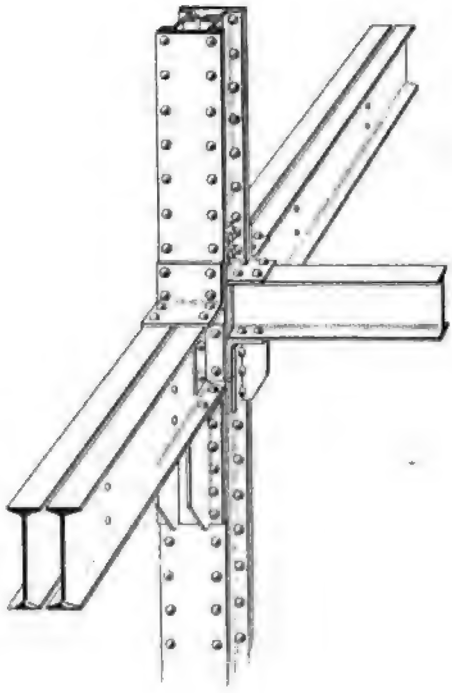


FIG. 2

the bracket. While this connection is adaptable to buildings of moderate height, it is inefficient in buildings of greater height than ten or twelve stories. In order to increase its rigidity, this connection is reenforced by a triangular plate known as a **gusset plate**, stiffened on the edges by angles. This type of bracing is shown in detail in Fig. 3. Where, in

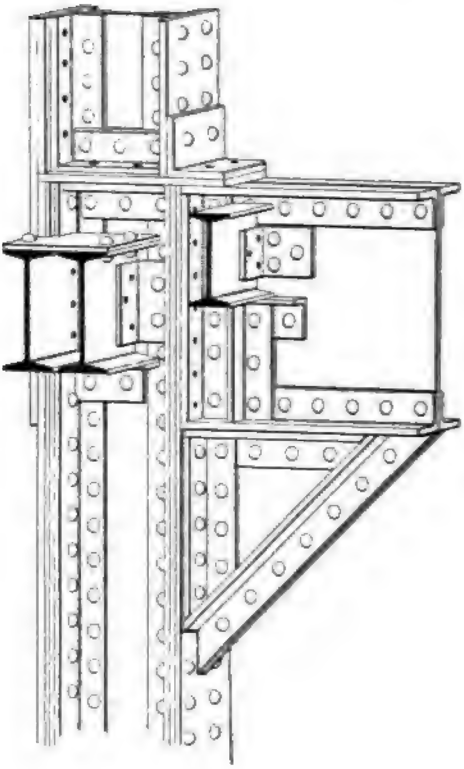


Fig. 3

the judgment of the designer, this connection fails to give sufficient rigidity, the **knee brace** shown in Fig. 4 is sometimes adopted. The gusset-plate form of bracing is much used in buildings whose height does not exceed fifteen stories, and whose least width is not less than one-fourth of the height. In many cases, the gusset plate is placed on

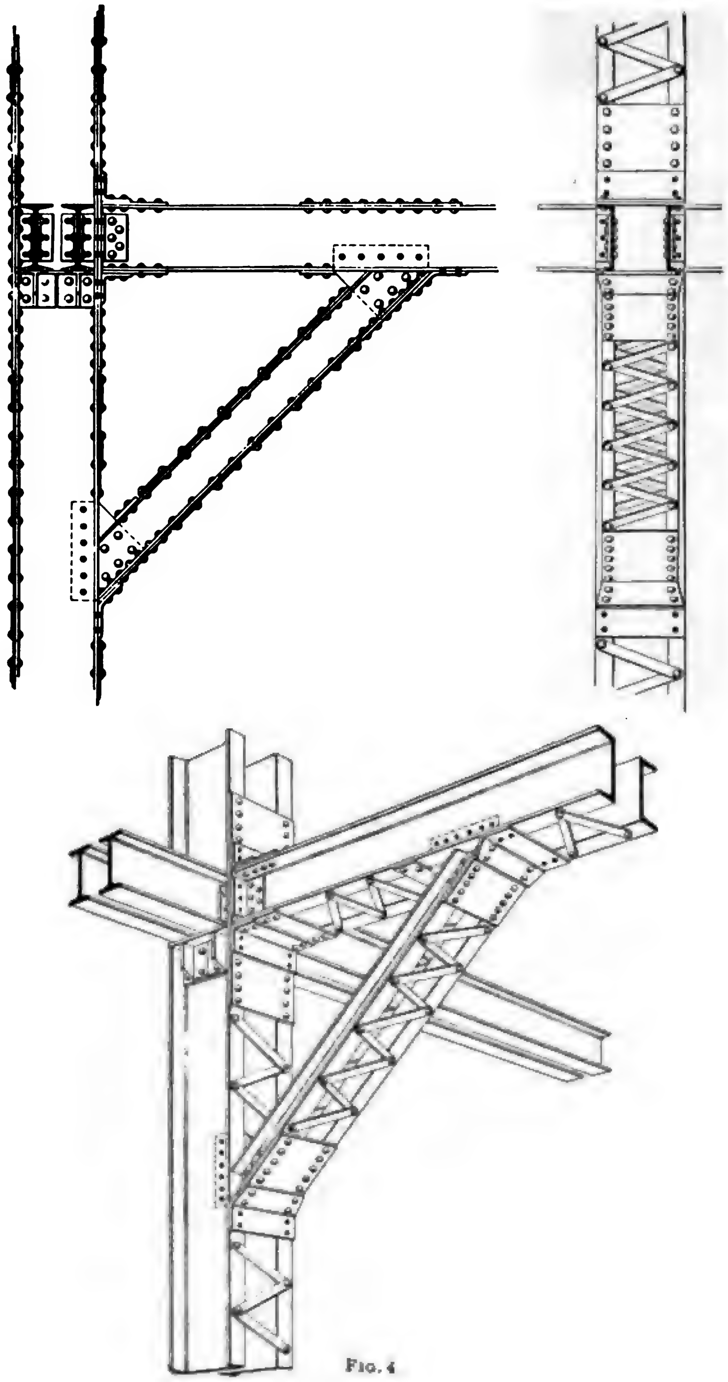


FIG. 4

both the upper and lower sides of the floor girder, as shown in Fig. 5, but this construction is not usual and possesses little advantage. The knee brace, shown in Fig. 4, subjects both the girders and the columns to great bending stresses, and is therefore objectionable.

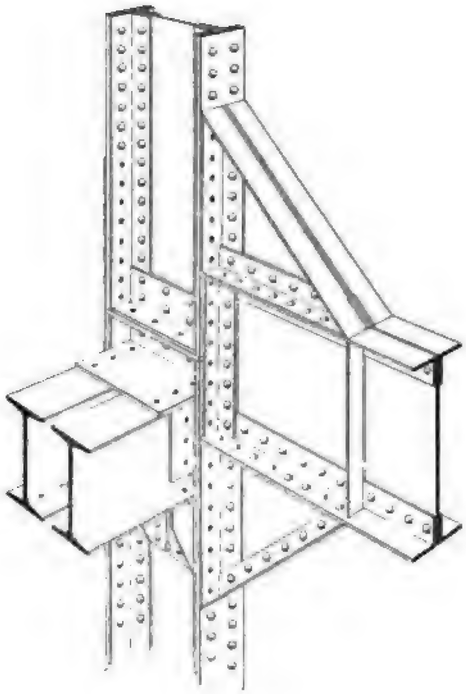


FIG. 5

5. Sway-Rod, or Diagonal, Bracing.—This type of wind bracing is usually constructed as shown in Fig. 6 (*a*). The diagonal bars *a*, *a* are tension bars that connect the top of one column with the bottom of the adjacent one; they prevent any distortion of the panel or frame formed by the floor girders and the columns. Fig. 6 (*b*) is a perspective view of the girder connection; it also shows the method of connecting the bars with the girder. Only one diagonal at a time is subjected to stress, for with the distortion of the frame,

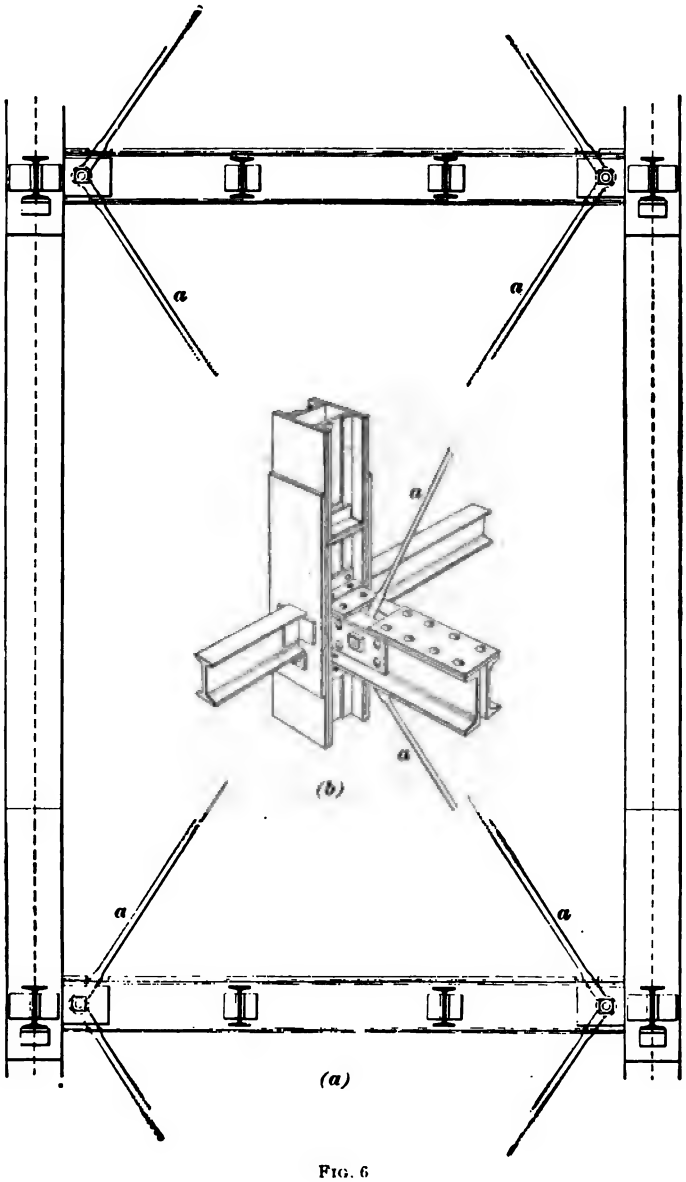


FIG. 6

the tendency is to lengthen the one diagonal and shorten the other. Since, therefore, they are tension bars and offer no resistance to compression, one-half of the entire bracing is useless when the wind blows on one side of the building. This condition is reversed, however, when the wind acts on

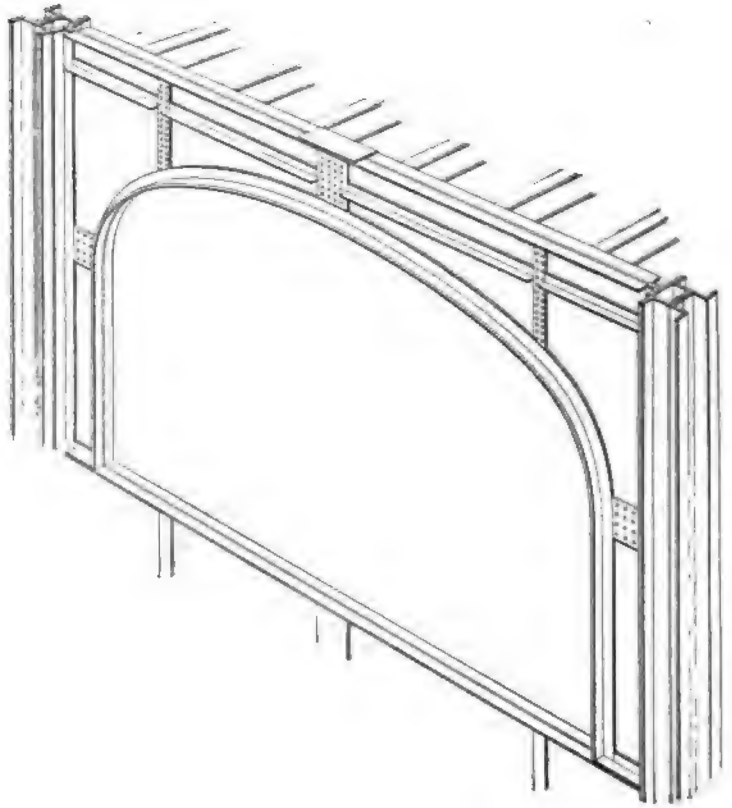


FIG. 7

the other side; then the member that was active is passive and the member that was previously useless comes into play. Excepting the gusset-plate bracing, the sway-rod, or diagonal, bracing is the most economical and reasonable. However, it has one serious objection in that it interferes

th door or window openings and must be concealed in rtitions. When, however, it is necessary to introduce a or or window opening in a partition or wall where the acing occurs, it is sometimes possible to introduce a agonal that will resist both compression and tension. In is case, the diagonal that is in the way of the opening may e omitted in that particular panel.

6. Portal Bracing.—This method of bracing is an xpensive form of construction, though it is often used in tall uildings. It is really a modification of the gusset brace nd consists of an arch built up of steel plates and angles xtending between the columns and the floor girders. Its usual type is shown in Fig. 7. It has the advantage of being particularly adaptable to architectural treatment. Where the portal bracing is used in the place of a girder or floorbeam, that is, where a steel arch is employed to carry the floor load as well as to provide lateral rigidity, the cost of this type of construction is somewhat lessened and is further decreased when the metal in the vertical portions of the portal is figured with the column section to resist the compression produced by the floor loads. This is doubtful construction and is not usually employed.

A modification of the portal bracing is known as the *portal strut*, which is a lattice girder placed between the two columns, as shown in Fig. 8 (*a*); it is more often used in steel-mill construction than in high-building work. The plan view, Fig. 8 (*b*), illustrates the construction of the column. In Fig. 9 is shown the complete girder with its connections to both columns.

7. It is usual in designing the wind bracing in tall buildings to carefully consider the plan and to place the bracing in the bays where it will be the most effective, but where it will not interfere with the architectural treatment. Buildings are seldom braced longitudinally, for they are usually narrow and of a considerable length. Where the building is 100 feet or more in length, it is considered good practice to provide bracing between two sets of the columns, as at a, a', b, and b',

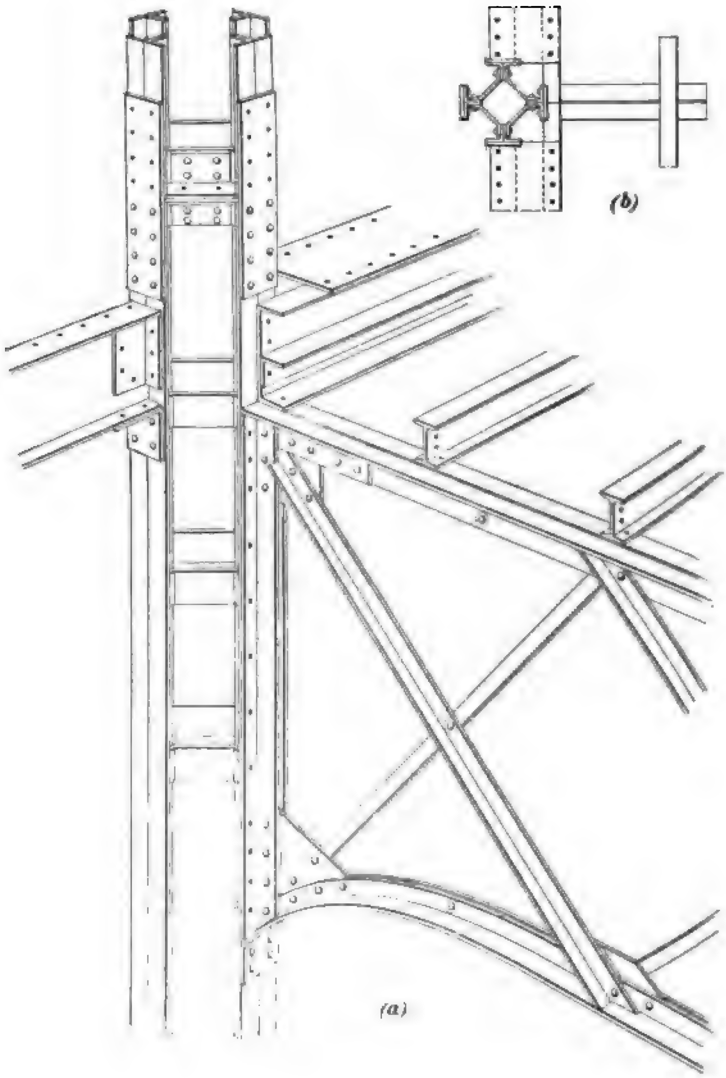


FIG. 8

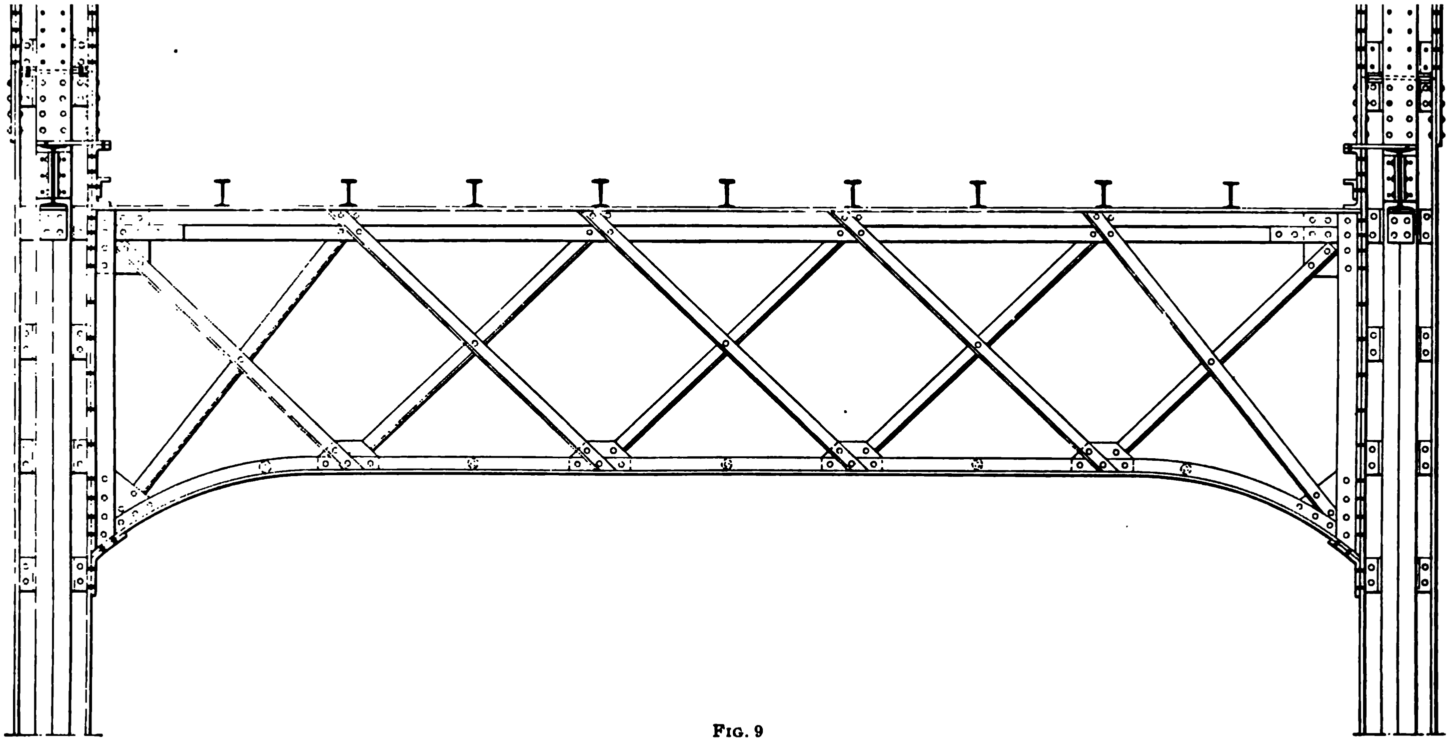


FIG. 9

Fig. 10. Should the building be of great length, two or more systems of bracing may be used, and frequently two sets of columns adjacent to each other are provided with bracing, as at

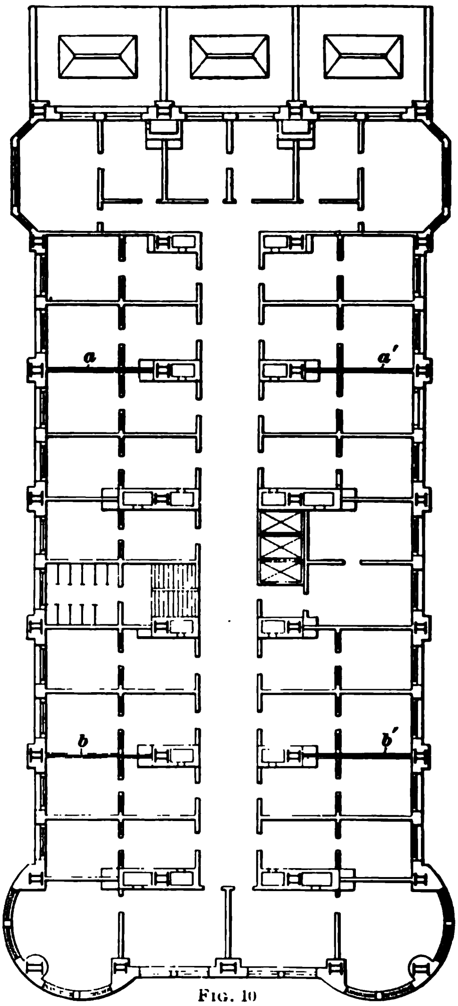


Fig. 10

a, a', b, and b', Fig. 11. The plan of the building has considerable to do with the location of the bracing, for the engineer's construction must be subservient to the architect's plan.

Sufficient bracing must, in all cases, be provided to resist the maximum pressure of 30 pounds per square foot of surface, and where the building offers a great surface—front and rear—it must be braced not only across, but also longitudinally. It is deemed unnecessary by some authorities to provide wind bracing for any area that may be sheltered by other buildings or for an area that abuts an adjacent wall. The conservative practice,

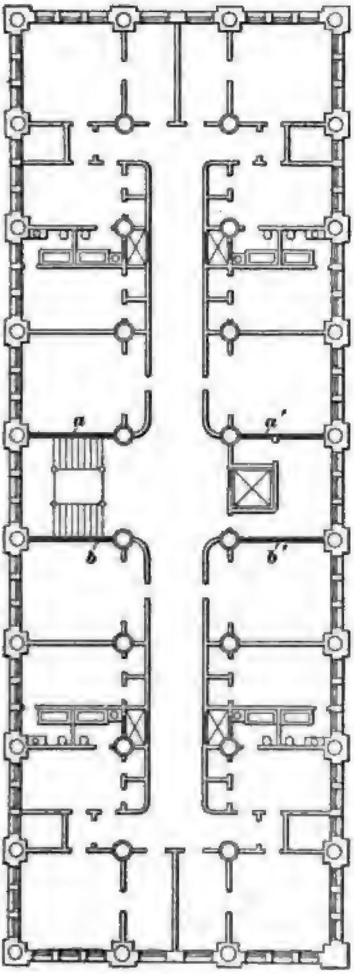


FIG. 11

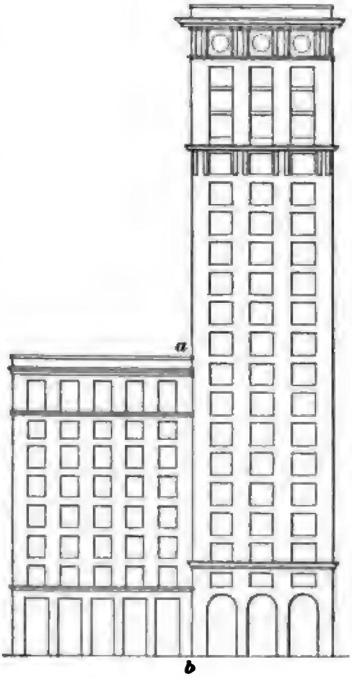


FIG. 12

however, is to provide wind bracing for the entire area offered by the side of the building, as a great fire or the removal of adjacent buildings may expose the area that was before

completely protected. For instance, in Fig. 12 is shown a tall building completely protected by the one next to it up to the height of ten stories. It is a question, in such an instance, that must be decided by the judgment of the designer, whether wind bracing should be so designed as to provide for a probable pressure on the portion of the wall from *a* to *b*, protected by the building to the left.

DETERMINATION OF STRESSES FOR BRACING HIGH STRUCTURES

RESULTANT PRESSURE AT EACH TIER OF BRACING

8. In order to determine the stresses in any system of wind bracing, it is necessary to find the area of exposed wall that must be provided for by the bracing. Even though they are only screen or curtain walls, heavy masonry end walls can be considered as sustaining a portion of the wind pressure. In the plan shown in Fig. 13, the system of bracing is shown at a, a', b, and b'. A single set of bracing may be considered as subjected to the wind pressure on an area equal to the height of the wall multiplied by one-half the distance between the two sets of wind bracing and one-half the distance from the set in question to the end wall, or by the distance c. A more conservative method would be to provide for the entire length of the wall in designing the wind bracing, but it must be left to the judgment of the designer whether this shall be done or whether the end walls shall be considered as offering sufficient rigidity against the wind pressure on a portion of the side wall.

On this point the building laws in several cities are clear, allowing that any end or partition wall can provide rigidity for the bracing of a portion of the side wall. The New York building law is not explicit, however, when it says: "In all structures exposed to the wind, if the resisting moment of the ordinary materials of construction, such as

masonry, partitions, floors, and connections is not sufficient to resist the moment of distortion due to wind pressure taken in any direction on any part of the structure, additional bracing shall be introduced sufficient to make up the difference in the moments." This law cannot be otherwise than uncertain, from the fact that the rigidity offered by these factors is indeterminate and only heavy masonry end walls or brick partition walls can enter into the problem, where a certain reasonable amount of the area of the side wall can be assumed as being taken care of by these means.

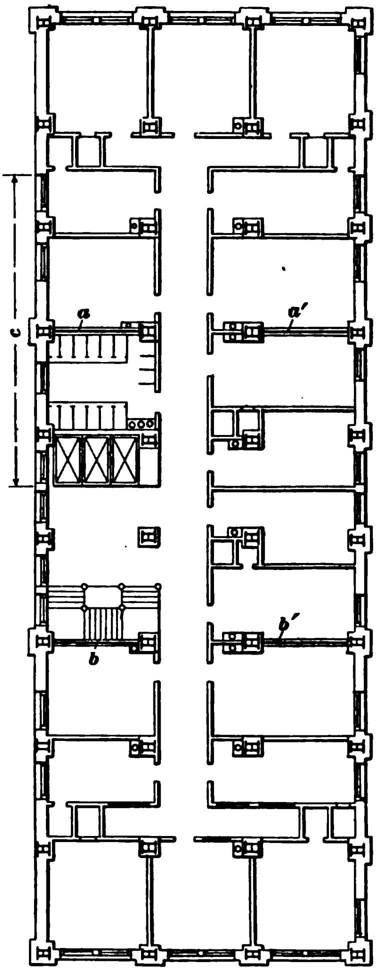


FIG. 13

9. After the width or length of the surface exposed to the wind, that is to be reenforced by one system of bracing, has been determined, the exposed area to be taken care of by each tier of columns, with its connecting bracing, may be found, and the resultant of the horizontal wind pressure at each floor level ascertained. This resultant wind pressure is not the force due alone to the wind pressure on the area of wall included between the floor level in question and the one above, but it is the amount of this pressure added to the resultant pressure at the floor above; in other words, it is the pressure on the surface included between the floor in

question and the top of the building. This is clearly understood when the building is considered as a cantilever beam supported at the foundations and loaded throughout with a uniformly distributed load, which represents the uniform wind pressure on the surface of the exposed wall. In Fig. 14, the skeleton framework of the building rests on the foundations,

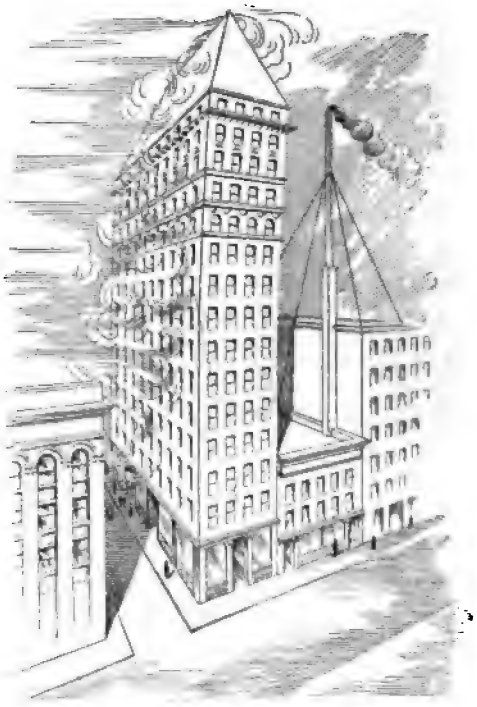


FIG. 14

to which it is securely held. The wind pressure is represented as acting on the left side of the building. The resultant of the wind pressure at each floor level is equal to the entire load included above the point in question; thus, if the building is regarded as a cantilever and the resultant pressure as the shear, the shear at any point is equal to the sum of the loads included between the end of the beam and the point in question.

Let the diagram, Fig. 15, represent the cross-section through a high and narrow building braced laterally against the wind pressure by diagonal, or sway, braces, as shown,

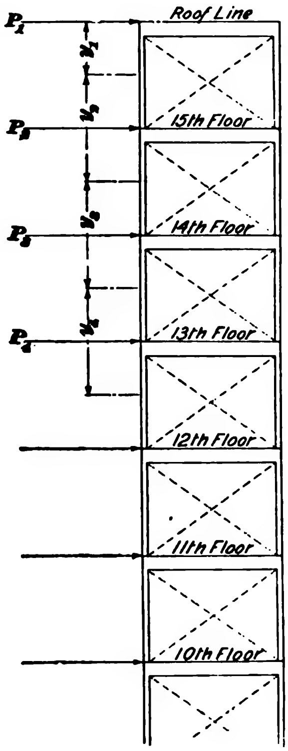


FIG. 15

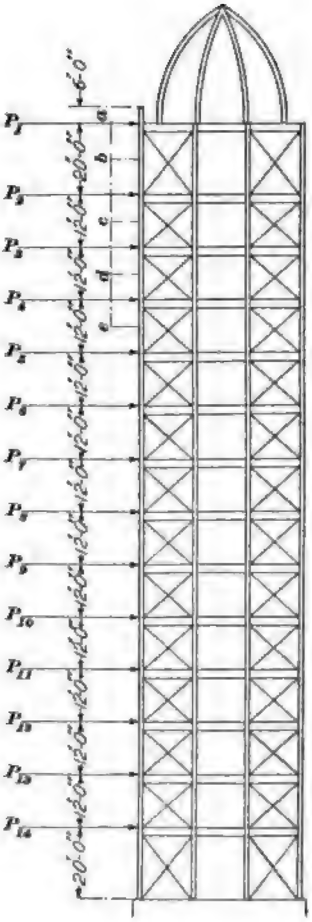


FIG. 16

and let P_1, P_2, P_3, P_4, etc. represent the resultant pressure at the roof line, the fifteenth, fourteenth, and thirteenth stories, respectively. P_1 will equal the pressure on the surface having a height of y_1, but P_2 will equal not only the pressure

on the surface having a height of y_2 but will include the pressure P_1. P_3 will be the sum of P_2 and P_1, and the pressure on a surface having a height of y_3, while the resultant pressure at P_4 will equal the pressure on the surface having a height of y_4 plus the pressures P_1, P_2, and P_3; so that if the product of the width of the exposed area and the normal pressure of the wind per square foot of surface is represented by x, the resultant pressure, or horizontal shear, as it may well be termed, at the roof and the several stories can be tabulated as follows:

$$P_1 = xy_1 \qquad P_3 = xy_3 + P_2$$
$$P_2 = xy_2 + P_1 \qquad P_4 = xy_4 + P_3$$

This notation can be carried to the bottom of the structure, or to the floor at the level of the roof of the adjacent building, should there be one. If such protection exists and it has been decided that the wind bracing need not be provided for the unexposed portion of the side walls, the bracing has no additional stress below the roof of the next building, but is designed to carry the last resultant pressure through all the tiers below that point to the foundation.

10. Assume that it is required to determine the resultant horizontal wind pressure at each of the floors of the office building shown in Fig. 16. The pressures to be determined are P_1, P_2, P_3, P_4, etc. The width of the area to be braced is 40 feet, while the dimensions between stories are as shown in the figure. P_1, or the pressure at the roof line, includes the wind pressure on the dome and the parapet, or cornice, which is assumed at 3,000 pounds, though this pressure may be in each case approximately calculated by considering the amount of the exposed area that would reasonably be sustained by the roof beams, and the pressure on the area included between a and b; P_2, P_3, and P_4 are the resultant pressures obtained by adding all the pressures above to the amount of the pressure on the area included between b and c, c and d, and d and e, respectively. Observing the notation described in Art. 9, the loads at the several floors in this building are as follows, when the normal wind

pressure, that is, the maximum horizontal pressure on the vertical surface, equals 30 pounds per square foot:

$$
\begin{aligned}
P_1 &= 40 \times 16 \times 30 + 3{,}000 = 22{,}200 \\
P_2 &= 40 \times 16 \times 30 + 22{,}200 = 41{,}400 \\
P_3 &= 40 \times 12 \times 30 + 41{,}400 = 55{,}800 \\
P_4 &= 40 \times 12 \times 30 + 55{,}800 = 70{,}200 \\
P_5 &= 40 \times 12 \times 30 + 70{,}200 = 84{,}600 \\
P_6 &= 40 \times 12 \times 30 + 84{,}600 = 99{,}000 \\
P_7 &= 40 \times 12 \times 30 + 99{,}000 = 113{,}400 \\
P_8 &= 40 \times 12 \times 30 + 113{,}400 = 127{,}800 \\
P_9 &= 40 \times 12 \times 30 + 127{,}800 = 142{,}200 \\
P_{10} &= 40 \times 12 \times 30 + 142{,}200 = 156{,}600 \\
P_{11} &= 40 \times 12 \times 30 + 156{,}600 = 171{,}000 \\
P_{12} &= 40 \times 12 \times 30 + 171{,}000 = 185{,}400 \\
P_{13} &= 40 \times 12 \times 30 + 185{,}400 = 199{,}800 \\
P_{14} &= 40 \times 16 \times 30 + 199{,}800 = 219{,}000
\end{aligned}
$$

The thrusts acting on the side of the building are therefore the vertical resultants of the wind pressure at each story; or, if the building is regarded as a cantilever, they represent the shear at these several points when the building is considered as secure at the foundation. Whatever form of bracing is used, these calculations must be made before the stresses in the members can be obtained.

EXAMPLES FOR PRACTICE

1. The width of the area requiring wind bracing in a seventeen-story building is 40 feet and the height of each story is 13 feet 6 inches; provided that the assumed wind pressure per square foot of vertical surface is 25 pounds and that the wind pressure on the top tier of beams supporting the roof, due to the pressure on the parapet, one-half of the upper story wall surface, and the superstructure on the roof, is 12,000 pounds, what will be the resultant wind pressure at the fourth, eighth, and twelfth stories?

Ans. { Fourth story, 201,000 lb.; Eighth story, 147,000 lb.; Twelfth story, 93,000 lb. }

2. A covered tower of the skeleton-construction type is 200 feet high and 40 feet square outside and the walls are merely curtain walls 8 inches thick supported on the frame, so that the steelwork of the tower must resist the entire wind pressure; provided the framework is in 10 tiers and the allowable horizontal wind pressure per square foot of vertical surface is 40 pounds, and also that the horizontal wind

pressure on the roof of the tower, transmitted to the top tier of beams from a superstructure, and on one-half of the wall surface between the top tier and the next tier below, amounts to 24,000 pounds, what will be the resultant wind pressure with the wind blowing normal to one of the sides, at the third, fifth, and eighth tiers of beams from the bottom?

Ans. { Third story, 248,000 lb. Fifth story, 184,000 lb. Eighth story, 88,000 lb.

GUSSET-PLATE AND KNEE BRACING

11. The gusset-plate type of wind bracing is similar in its action to knee bracing and the method of obtaining the stresses created in the columns and girders is the same for both of these types. In considering the gusset-plate type of bracing, shown in Fig. 17 (*a*), it is usual to neglect the gusset plate in the consideration and to proportion the stiffening angles at the edge of the plate for the entire stress, though where economy demands it and in the judgment of the designer it is advisable, the portion of the gusset plate between the reenforcing angles, as shown in view (*b*), may be considered as a part of the section of the angles, or of the strut composed of the reenforcing angles.

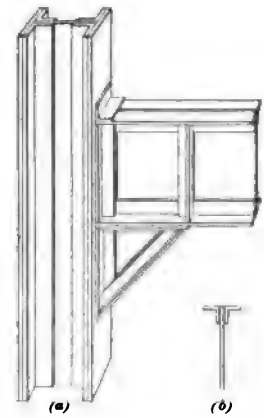


FIG. 17

A diagrammatic representation of the gusset or knee

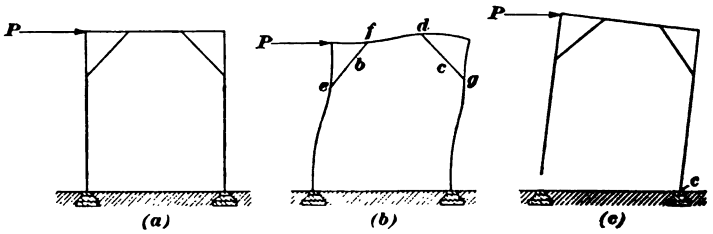


FIG. 18

braces subjected to wind pressure is shown in Fig. 18 (*a*), while in (*b*) is shown the same structure distorted by the

orizontal resultant of the wind pressure acting on the left-hand side and at the junction of the floor girder with the column; the resultant pressure in each case is represented by P.

In the design of all framed structures it is customary to consider the joints as being hinged or pin-connected because the rigidity of a structural joint is practically indeterminate, though in wind bracing the column is assumed to be a rigid member, resisting the thrust of the lower end of the knee brace by its transverse strength.

12. Columns Secured at Base.—In tall buildings, the columns are always considered as being fixed at the ends, so that the wind pressure at the top of the knee-braced frame will create the distortion shown in Fig. 18 (b); with this distortion, stresses are created that produce tensile strains in the windward knee brace b, and compressive strains in the leeward knee brace c. The stresses transmitted by these braces pull and push against the girder, or floorbeam, and columns at the points e, f, d, and g, tending to produce the distortion shown and creating considerable bending moments in these members.

Also, the action of the pressure P at the top of the frame, as in Fig. 18 (a), tends to overturn the structure about the point c as shown in view (c). It may be seen from this that there is a lifting tendency on the column to the left and an equivalent compression tendency on the column to the right. Both columns must therefore be proportioned to resist this amount of tension and they must likewise be proportioned to resist this additional compression from the wind pressure.

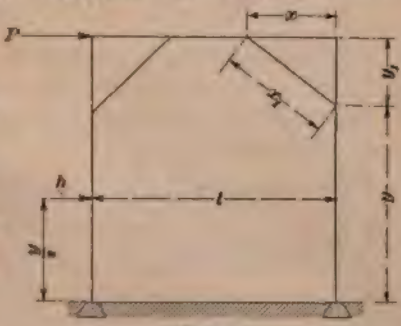


FIG. 19

13. The stresses in the several members of the knee-braced frame created by the horizontal resultant of the wind

pressure P acting at the top of the frame may be determined by the following formulas. The notation used is readily understood by reference to Fig. 19, in which the point of *inflexion*, or bending, is at h; in the derivation of the formulas, this point, instead of the joint at the base, is considered as hinged.

In knee braces, the tension or compression equals

$$P\left(\frac{1}{2}+\frac{y}{4y_1}\right)\frac{x_1}{x} \qquad (1)$$

In columns, the tension or compression equals

$$\frac{P\left(\frac{y}{2}+y_1\right)}{l} \qquad (2)$$

In girders, the tension or compression equals

$$P\left(1+\frac{y}{4y_1}\right) \qquad (3)$$

In columns, the bending moment equals

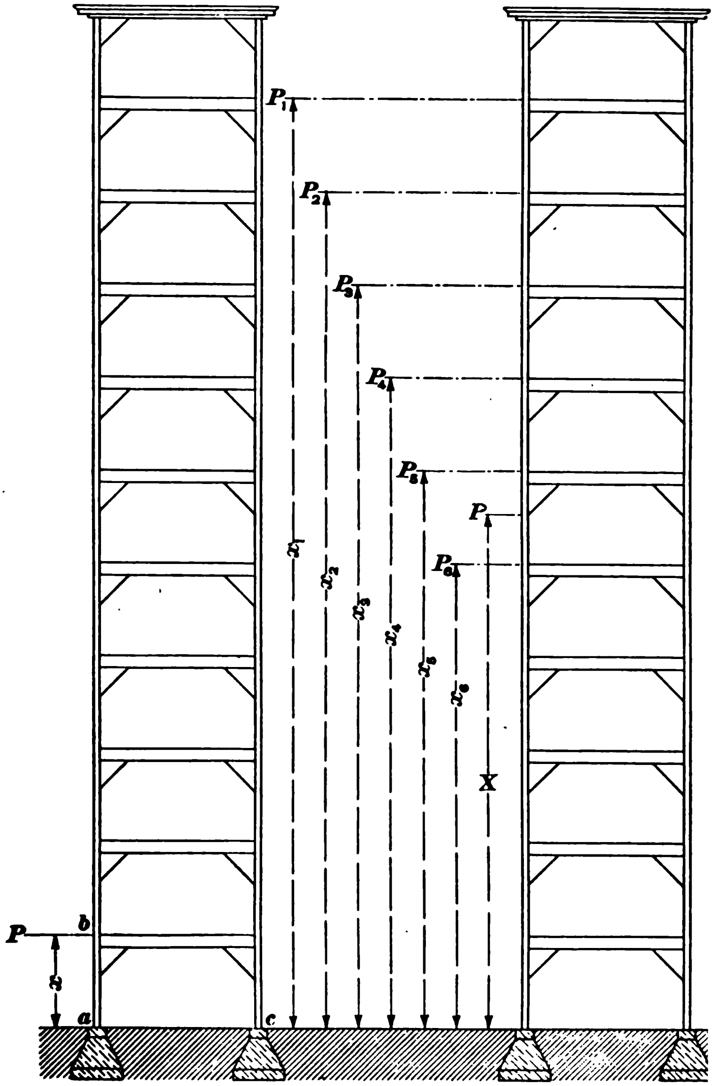
$$P\frac{y}{4} \qquad (4)$$

In girders, the bending moment equals

$$P\left(\frac{1}{2}-\frac{x}{l}\right)\left(y_1+\frac{y}{2}\right) \qquad (5)$$

When substituting in the formulas for the bending moments, it is advisable to reduce the values of y, x, y_1, and l to inches, for then the result may be equated with the resisting moment of the members.

NOTE.—Formulas **2** and **7**, together with **17** (which determines the direct compressive and tensile stresses in the columns), apply only to a single tier of bracing on which other stories are not supported. All other formulas from **1** to **10** may be used for proportioning the knee-braced frame of a tall building. Where it is desired to find the direct tensile and compressive stresses created in the columns of a tall building, knee-braced at each story, divide the sum of moments of each individual shear about the foot of the leeward column by the distance from center to center of the columns forming the system. Notice particularly that there is a difference between the individual shears and the accumulated shear at each floor. The method of finding the accumulated shear, or the resultant P, at each floor level was described in Art. **9**; the individual shear is simply the load or wind pressure on one-half of the panel above and below the floor in question.



(a) (b)

FIG. 20

This statement is more clearly understood by reference to Fig. 20, where (*a*) shows a frame in which P equals the accumulated shear at the bottom tier of floorbeams. The value of P, with the exception of the pressure on one-half of the panel *a b*, is equal to the entire wind pressure. It is quite evident that P multiplied by its lever arm x will not give the maximum bending moment about the point c; but that this maximum moment will be created by PX in Fig. 20 (*b*) is evident, or, barring the discrepancy due to the pressure on one-half of the panel *a b*, the maximum moment will be created by the following sum of moments: $P_1 x_1 + P_2 x_2 + P_3 x_3$, etc.

EXAMPLE.—Determine the tension and compression in the knee braces, girders, and columns, and the bending moments in the columns and girders in the structural frame or panel shown in Fig. 21. This diagram represents the uppermost tier in a building of the skeleton-construction type; the resultant wind pressure at the top of the frame has been found, by the method described in Art. **10**, to equal 20,000 pounds.

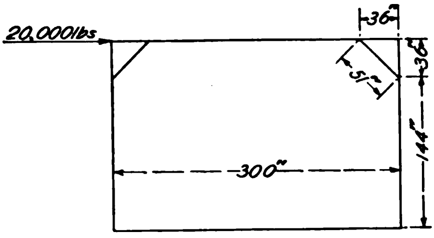


FIG. 21

SOLUTION.—For the tension and compression stresses in the brackets, or knee braces, the columns, and girders, formulas **1**, **2**, and **3** are used, while for the bending moments on the girders and columns, formulas **4** and **5** are employed. From the problem, the value of P in each of these formulas is 20,000, and the values of y, y_1, x, x_1, and l are equal, respectively, to 144, 36, 36, 51, and 300 in. Then, by substitution, the stresses in the several members are as follows:

In the knee braces, the tension or compression equals

$$20{,}000\left(\frac{1}{2} + \frac{144}{4 \times 36}\right)\frac{51}{36} = 42{,}500 \text{ lb. Ans.}$$

In the columns, the tension or compression equals

$$20{,}000 \times \frac{\left(\frac{144}{2} + 36\right)}{300} = 7{,}200 \text{ lb. Ans.}$$

In the girders, the tension or compression equals

$$20{,}000\left(1 + \frac{144}{4 \times 36}\right) = 40{,}000 \text{ lb. Ans.}$$

In the columns, the bending moment equals

$$20{,}000 \times \frac{144}{4} = 720{,}000 \text{ in.-lb. Ans.}$$

In the girders, the bending moment equals

$$20{,}000\left(\frac{1}{2} - \frac{36}{300}\right)\left(36 + \frac{144}{2}\right) = 820{,}800 \text{ in.-lb. Ans.}$$

14. Columns Insecure at the Base.—Where wind acing is to be provided for low sheds, mill buildings, or ops, as in Fig. 22, it is customary to regard the columns being hinged rather than fixed at the ends. This assumpon is made because the foundations usually provided under

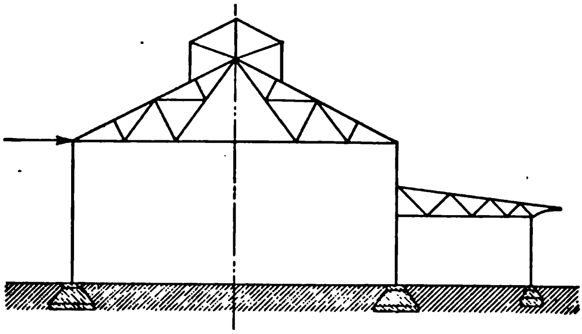


Fig. 22

the columns for such buildings are totally inadequate to offer the required resistance. When the columns are hinged or free to give at the ends, that is, are not rigidly secured at the ends, the stresses throughout the knee-braced frame will be increased, for the value $2y$ must be substituted for the value of y in formulas **1** to **5**, when they are as follows:

In the knee braces, the tension or compression equals

$$P\left(\frac{1}{2}+\frac{y}{2y_1}\right)\frac{x_1}{x} \qquad (6)$$

In the columns, the tension or compression equals

$$\frac{P(y+y_1)}{l} \qquad (7)$$

In the girders, the tension or compression equals

$$P\left(1+\frac{y}{2y_1}\right) \qquad (8)$$

In the columns, the bending moment equals

$$P\frac{y}{2} \qquad (9)$$

In the girders, the bending moment equals

$$P\left(\frac{1}{2}-\frac{x}{l}\right)(y_1+y) \qquad (10)$$

EXAMPLES FOR PRACTICE

1. The horizontal resultant of the wind pressure at the tenth story of an office building is 40,000 pounds. The distance between the columns of the knee-braced frame is 20 feet while the height of the columns is 18 feet; the distance from the foot of the column to the knee connections is 13 feet, and the knee braces are at an angle of 45°: (*a*) What will be the stress in the knee braces? (*b*) What will be the bending moment in the columns?

Ans. $\begin{cases} (a)\ 65{,}044 \text{ lb.} \\ (b)\ 1{,}560{,}000 \text{ in.-lb.} \end{cases}$

2. From the data given in example 1, determine: (*a*) the bending moment due to the wind load on the floor principal or girder; (*b*) the amount of tension or compression to which the columns are subjected from the wind load.

Ans. $\begin{cases} (a)\ 1{,}380{,}000 \text{ in.-lb.} \\ (b)\ 23{,}000 \text{ lb.} \end{cases}$

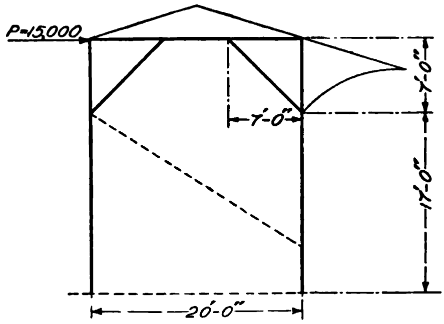


FIG. 23

3. Determine all the stresses in the structural members of the grand stand shown in Fig. 23, assuming the columns to be rigidly secured at the ends. The wind pressure is considered as applied at *P* and for one bay amounts to 15,000 pounds. All the stress is considered as being resisted by the frame of the structure, which is shown by the heavy lines.

Ans. $\begin{cases} \text{Tension or compression in knee brace, } 23{,}486 \text{ lb.} \\ \text{Tension or compression in column, } 11{,}625 \text{ lb.} \\ \text{Tension or compression in girder, } 24{,}107 \text{ lb.} \\ \text{Bending moment in columns, } 765{,}000 \text{ in.-lb.} \\ \text{Bending moment in girders, } 418{,}500 \text{ in.-lb.} \end{cases}$

SWAY, OR DIAGONAL, BRACING

15. The method of determining the stresses for diagonal, or sway-rod, wind bracing is the same as that employed in analyzing a cantilever trussed girder. The wind bracing in the building, as previously stated, is considered as a cantilever secured at the foundation and loaded on one side with the wind pressure. The first step in determining the stresses is to find the accumulated, or resultant, wind pressure at each tier of floorbeams, which was described in Arts. **9** and **10**.

'ig. 24 is shown a diagonal, way-braced, frame in which resultant pressures at the eral tiers are designated by , P_2, P_3, P_4, P_5, P_6, and P_7. is evident, by inspection, that th the wind on the left-hand le of the structure, as shown, e diagonal tension rods iown dotted are useless, and iat the entire stress is trans- nitted through the tension ods represented by the heavy ines. This is the only system of bracing that does not create bending stresses in the floor principals and in the columns. The stresses created are tensile in the diagonals, compressive and tensile in the columns, and compressive in the floor girders, or principals. The horizontal resultant of the wind pressure at each floor is the amount of compression in the floor girders, or principals, so that the stresses in these members are as follows:

$$a\,b = P_1 = 10{,}000$$
$$c\,d = P_2 = 20{,}000$$
$$e\,f = P_3 = 30{,}000$$
$$g\,h = P_4 = 40{,}000$$
$$i\,j = P_5 = 50{,}000$$
$$k\,l = P_6 = 60{,}000$$
$$m\,n = P_7 = 70{,}000$$

16. The ratio of the stresses in the diagonals to the stresses

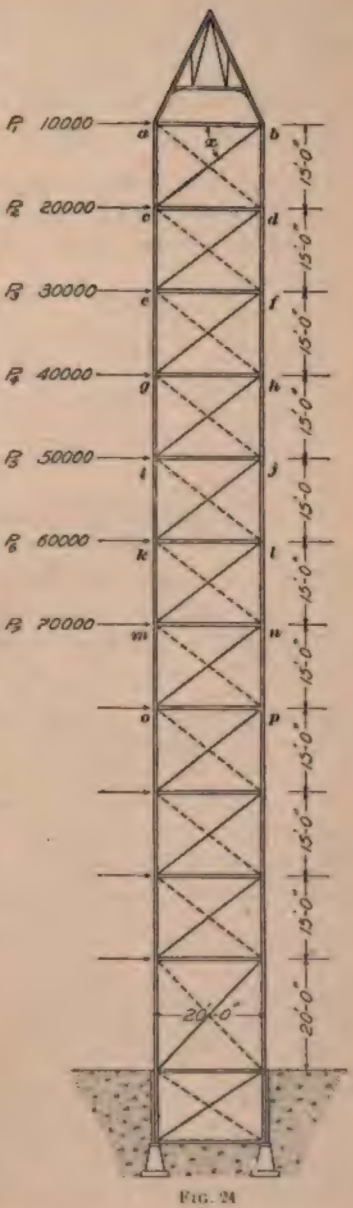


FIG. 24

in the horizontal members or girders is equal to the ratio between their respective lengths. The length of the diagonal bc may be found by means of the equation $bc = ab \times \sec x$. If the horizontal member ab is assumed to be of unit length, the length of bc will be equal to the secant of the angle x. The secant of the angle equals $\frac{bc}{ab}$, but as

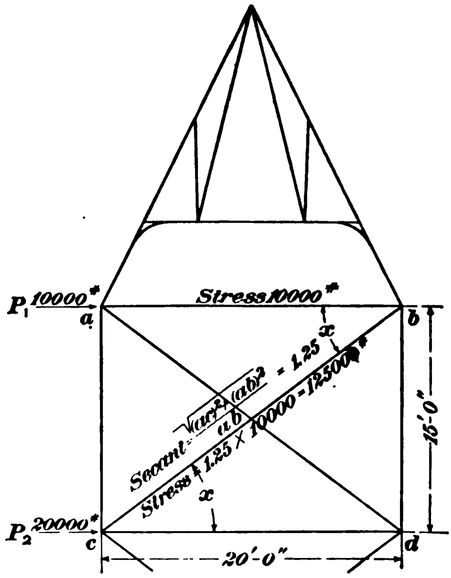


FIG. 25

bc is unknown it may be found by using the formula $bc = \sqrt{ab^2 + ac^2}$. On substituting this value of bc in the preceding equation we have, as shown in Fig. 25, $\sec x = \frac{\sqrt{ab^2 + ac^2}}{ab}$, and, after inserting the values of ab and ac, it is found that $\sec x = \frac{\sqrt{20^2 + 15^2}}{20} = 1.25$, indicating that the relative sizes of bc and ab are as $1.25 : 1$. Consequently, since the stresses are proportional to the lengths of the respective members, the stress in the member bc equals the stress in the member ab multiplied by 1.25, or 12,500 pounds.

'he formula $bc = ab \times \sec x$ may be used for finding the esses in the diagonals without first considering their gths by substituting for the length of ab the stress in ne. The stress in any of the diagonals will then be:

$$P \sec x \qquad (11)$$

which P = resultant wind pressure at the top of diagonal;
x = angle between diagonal and horizontal.

'hen the stresses for the frame shown in Fig. 24 are as ows:

$$\begin{aligned}
cb &= 10{,}000 \times 1.25 = 12{,}500\\
ed &= 20{,}000 \times 1.25 = 25{,}000\\
gf &= 30{,}000 \times 1.25 = 37{,}500\\
ih &= 40{,}000 \times 1.25 = 50{,}000\\
kj &= 50{,}000 \times 1.25 = 62{,}500\\
ml &= 60{,}000 \times 1.25 = 75{,}000
\end{aligned}$$

.**7.** The columns are subjected to compression and sion in the same manner as the knee-braced frame, and stresses in the columns at each floor level, due to the ar at the floor level above, are proportional to the horital resultant of the wind stress as the length of the column to the length of the horizontal member. If, in the mula $\tan x = \frac{ac}{ab}$, ab is assumed to be of unit length, ac l equal $\tan x$, Fig. 25. The tangent of the angle x is ial to $\frac{15}{20}$ = .75, indicating that $ac : ab = .75 : 1$.

)n substituting for ab the horizontal resultant of the id pressure P, the stress in any of the columns, directly e to the resultant wind pressure at the floor above, may found by means of the formula

$$P \tan x \qquad (12)$$

'n this instance, $\tan x = .75$; therefore, the stress in umn $ac = 10{,}000 \times .75 = 7{,}500$ pounds.

The amount, 7,500 pounds, just obtained is the compresn in the column ab and the tension in the column ac, ich, as seen in Fig. 24, are the uppermost ones of the me. The columns in the next tier beneath, designated

by *ce* and *df*, are subjected not only to the tension and compression created by the resultant pressure P_2, but must in addition sustain the compression and tension that is transmitted from the columns in the tier above. The stresses created in each tier of columns by the resultant pressure at the floor above may be called the *increment wind stresses*, so that the tension or compression in any tier of columns for the frame shown in Fig. 24 is found from the following notation:

COLUMNS	RESULTANT	TAN x	INCREMENT	STRESS
ac, *bd*	P_1, or 10,000	× .75 =	7,500	7,500
ce, *df*	P_2, or 20,000	× .75 =	15,000	22,500
eg, *fh*	P_3, or 30,000	× .75 =	22,500	45,000
gi, *hj*	P_4, or 40,000	× .75 =	30,000	75,000
ik, *jh*	P_5, or 50,000	× .75 =	37,500	112,500
km, *lm*	P_6, or 60,000	× .75 =	45,000	157,500
mo, *np*	P_7, or 70,000	× .75 =	52,500	210,000

In this calculation, the values under the column headed Stress are obtained by adding to the increment the value of the stress in the line above. Thus, the stress in each of the columns *ik* and *jh* is 75,000 + 37,500 = 112,500 pounds.

18. Though the method of determining all the stresses in a sway-braced frame has been given, it is possible that the calculations may be made clearer by an algebraic analysis of the stresses and the consequent derivation of the simple formulas for the stresses in the several members. Referring to Fig. 26, it will be observed that, in the sway-braced frame shown, the diagonals are not secured to the frame at the intersection of the axes of the horizontal floor members and the vertical column members, but are secured to the horizontal member at some distance from the axis of the columns. This feature does not influence the direct stresses in the members, though the eccentric loads on the column, due to the vertical components of the stresses in the sway rods secured at *a* and *b*, must be taken into account in the design of the column.

In Fig. 26, p will represent not the resultant shear at c, but the wind load that is transmitted to this panel point c from one-half of the panel $a\,c$ and one-half the panel $c\,e$, while P will equal the horizontal component of the stress produced in the sway or diagonal rod $c\,b$ by the wind load at panel point a, which stress will be designated by D. The entire stress in the member $c\,d$ will then equal $P + p$.

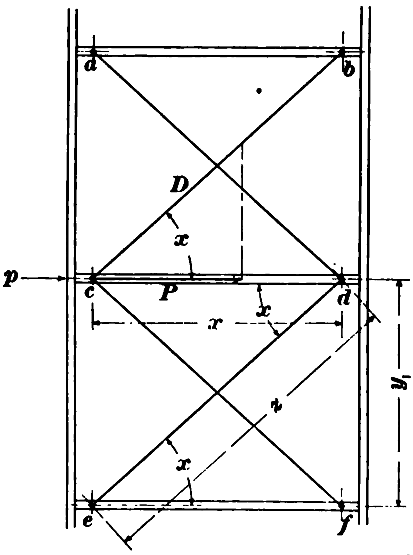


FIG. 26

The stress in the columns $c\,e$ and $d\,f$ is equal to the vertical components of the stress in the sway rod $c\,b$ plus the additional vertical component of the stress in the sway rod $c\,d$ induced by the panel load p. This stress may therefore be expressed by $(P + p)\tan x$; or, if the horizontal distance between the centers of pins connecting the sway rods to the beams is denoted by x, and y_1 represents the vertical distance between the pins on adjoining beams, x and y_1 may be considered as the active lengths of the respective members, and as $\frac{y_1}{x} = \tan x$, the expression $(P + p)\tan x$ may be changed to $\frac{(P + p)y_1}{x}$, which will give, by calculation, the same result as though the resultant shear due to the wind pressure, accumulated at the panel point c, was multiplied by $\frac{y_1}{x}$, or by the tan x. The stress in the tension member, or sway rod, $e\,d$ is proportional to the stress in the member $c\,d$, as the distance $c\,d$ is to the length of $e\,d$, or, trigonometrically, the stress in the member equals $(P + p)\sec x$, which is the

same as $\frac{\sqrt{y_1^2 + x^2}(P + p)}{x}$; or, if the length ed is designated by z, this stress may be expressed as $\left(\frac{P+p}{x}\right)z$.

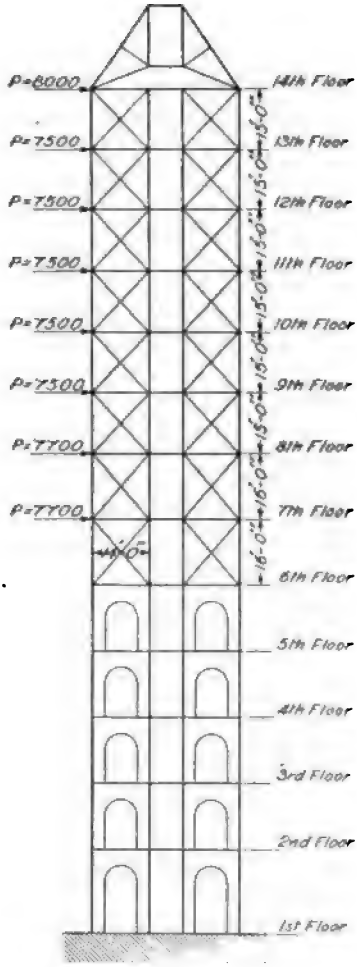


FIG. 27

These equations are convenient for calculating the stresses existing in the several members of a sway-braced frame and may be arranged as follows:

In the sway rods, the tension equals

$$\frac{\sqrt{y_1^2 + x^2}(P + p)}{x} \qquad (13)$$

In the columns, the tension or compression equals

$$\frac{(P+p)y_1}{x} \qquad (14)$$

In the horizontal struts or floor girders, the compression equals

$$P + p \qquad (15)$$

In these formulas, x and y_1 are made clear by reference to Fig. 26, in which P equals the shear from the loads above at the panel point under consideration, while p equals the panel load at the same place.

This completes the determination of all stresses. When the wind blows from the opposite side of the braced frame, the stresses in the diagonals are reversed, likewise in the columns, one set of diagonals becoming useless. If it is desirable to omit

one of the diagonal tension members in order that a space may be provided for door and window openings, the remaining member must be proportioned for both tension and compression.

EXAMPLES FOR PRACTICE

1. In Fig. 27 is shown the section through a tall office building, the upper floors of which are braced with sway rods or diagonal bracing. If the left-hand system of bracing is considered as sustaining all the wind pressure on that side of the building, what will be the compression in the floor members at the seventh, ninth, and eleventh floors?

Ans. { Seventh floor, 60,900 lb.
Ninth floor, 45,500 lb.
Eleventh floor, 30,500 lb.

2. Under the same condition of loading and construction as shown in Fig. 27, what will be the stress in the diagonals between: (*a*) the eleventh and tenth floors? (*b*) the seventh and sixth floors?

Ans. { Eleventh and tenth floors, 44,700 lb.
Seventh and sixth floors, 92,482 lb.

3. To complete the analysis of the stresses for the sway bracing shown in Fig. 27, determine the additional compression or tension that will be created by the wind pressure in the columns between: (*a*) the eighth and ninth floors? (*b*) the twelfth and thirteenth floors?

Ans. { Eighth and ninth floors, 48,750 lb.
Twelfth and thirteenth floors, 16,607 lb.

PORTAL BRACING

19. In the latticed-portal brace shown in Fig. 28, the stresses are similar to those existing in the knee-braced frame, with the exception that there is no bending moment created in the horizontal member connecting the tops of the columns. It is a system of bracing much used in bridge work and sometimes in buildings, though it is more often used in high one-story sheds and mills than in the type known as skeleton construction. When the diagonal members are considered as resisting tensile stress only, as is usually the case, the top and bottom chords of the horizontal members *a b* and *c d* are in either tension or compression. The posts or columns, as in all systems of wind bracing, may be subjected to either tensile or compressive

stress, due to the overturning tendency of the frame, as explained in Art. **12.**

The only bending moment created in this type of bracing is in the columns; this is maximum at the points *c* and *d*, and when the columns are fixed at the lower end it is calculated by formula **4.** The stresses in the several members of the lattice-portal bracing shown in Fig. 28 may be determined by the following formula, in which the dimensions required are designated by y, y_1, x, and l, which correspond to the dimensions similarly marked in the figure, and the resultant wind pressure at the top of the frame is designated by P. The formulas are based on the assumption that the columns or posts are fixed at the ends; if they are not secured at the ends, the value $2y$ must be substituted for y.

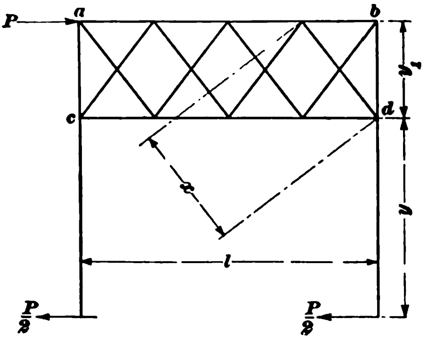


FIG. 28

In the top chord *a b*, tension or compression equals

$$P\left(1+\frac{y}{4y_1}\right) \qquad (16)$$

In the bottom chord *c d*, the tension or compression equals

$$P\left(\frac{1}{2}+\frac{y}{4y_1}\right) \qquad (17)$$

In the diagonals, the tension or compression equals

$$P\left(\frac{y_1}{2}+\frac{y}{4}\right)\frac{x}{l\,y_1} \qquad (18)$$

In the columns or posts, the tension or compression equals

$$\frac{P\left(y_1+\frac{y}{2}\right)}{l} \qquad (19)$$

In the columns or posts, the bending moment equals

$$P\frac{y}{4} \qquad (20)$$

n these formulas, in calculating the bending moment on columns, it is advisable to reduce the values of y, y_1, x, and l, given in the formulas, to inches, so that the result obtained will be in inch-pounds.

FIG. 29

EXAMPLE.—Determine the stresses in the several members of the lattice-portal brace frame shown in Fig. 29, provided that the horizontal wind resultant at the top of the frame is 23,000 pounds.

SOLUTION.—The values of y, y_1, x, and l are equal, respectively, to 16, 4, 5.6568, and 16 and the value of P from the problem is 23,000. On substi-uting in formulas **16** to **20**, the stresses in the several members are s follows:

In the top chord, or ab, the tension or compression equals

$$23{,}000\left(1+\frac{16}{4\times 4}\right)=46{,}000 \text{ lb.} \quad \text{Ans.}$$

In the bottom chord, or cd, the tension or compression equals

$$23{,}000\left(\frac{1}{2}+\frac{16}{4\times 4}\right)=34{,}500 \text{ lb.} \quad \text{Ans.}$$

In the diagonals, the tension or compression equals

$$23{,}000\left(\frac{4}{2}+\frac{16}{4}\right)\frac{5.6568}{16\times 4}=12{,}197.475 \text{ lb.} \quad \text{Ans.}$$

In the columns or posts, the tension or compression equals

$$\frac{23{,}000\left(4+\frac{16}{2}\right)}{16}=17{,}250 \text{ lb.} \quad \text{Ans.}$$

In the columns or posts, the bending moment equals

$$23{,}000\times\frac{16\times 12}{4}=1{,}104{,}000 \text{ in.-lb.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. On the top of four columns connected at their upper end by lattice-portal bracing is to be placed a cylindrical water tank 15 feet in diameter and 40 feet high. The distance from center to center of columns is 12 feet and the height of the column from the

foundation to the top of the bracing is 25 feet. The distance between the centers of the top and bottom chords of the lattice bracing is 3 feet and the top and bottom chords are divided horizontally into four panels. What will be the stresses in the several members of the frame when the tank is empty and when it is assumed that the resultant wind pressure acts at the top of the frame in a direction parallel with a horizontal line connecting two adjacent columns, all of the wind pressure against the frame itself being neglected? The effective wind pressure against its surface at the maximum of 30 pounds per square foot is only one-half as great as when the wind acts against a flat surface.

Ans. { Top chord, 25,500 lb.
Diagonals, 7,424 lb.
Tension in posts, 10,500 lb.
Bending moment, 594,000 in.-lb.

2. In a large machine shop, the columns supporting the track for the traveling crane are connected by lattice girders. The lateral pull at the top of the frame on a single set of columns due to starting and stopping is 10,000 pounds. The columns are 20 feet from center to center, 18 feet high, and the depth of the lattice bracing from center to center of chords is 4 feet, while the top and bottom chords are divided horizontally into 5 panels. What is: (*a*) the amount of the stresses created in the columns and (*b*) in the diagonal bracing due to the horizontal force exerted by the crane?

Ans. { (*a*) 13,000 lb.
(*b*) 9,192 lb.

---

20. The three elements in the type of bracing shown in Fig. 30 are the columns, the floorbeams or girders, and the portal, which is the reenforcing member that holds the floorbeams or girders and the columns rigidly in their respective positions and prevents any distortion of the rectangular frame or panel included between them. The stresses that exist in this type of portal bracing consist of direct compression and tension in the columns and the vertical portions of the portal; also, bending moments throughout certain portions of the portal, as well as vertical and horizontal shear throughout the frame. There is a tendency for the structure to revolve about the foot of the leeward column or portal with a moment at any floor equal to the moment of the resultant wind pressure on all the surface above the point in question.

This is more clearly understood by reference to Fig. 31, where the moment of the force P_1, which is the wind pressure

on the top story, is equal to its amount multiplied by the perpendicular distance between its line of action and the point c_1. If, therefore, P_1 equals the pressure on the top story and x_1 the lever arm or the perpendicular distance, its turning moment about c_1 equals $P_1 x_1$. Similarly, the pressure P_2, which is the entire horizontal wind pressure on the two top floors, that is, on the surface from s to s_2, creates a moment about c_2 equal to $P_2 x_2$, while the moment of P_3 about the point c_3 equals $P_3 x_3$, this moment being created by the pressure on the surface included between s and s_3. It will be noticed that in each case the pressure on the entire surface included between the top and the point of revolution around which the frame tends to overturn is considered, and that the line of action of the pressure is taken at the center of the surface, or, if the wind pressure is regarded as a uniformly distributed load, at the center of gravity of the load. The overturning tendency at the first story, or the base, of the building would evidently equal the wind pressure on the side of the building multiplied by one-half the height. This overturning moment at each floor creates tension in the windward column and compression in the leeward, and the theoretical amount of these direct tensile and compressive stresses is obtained by dividing

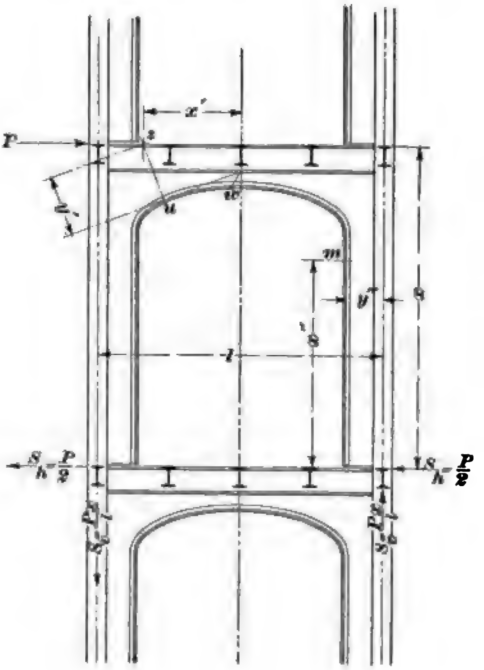


FIG. 30

the overturning moment at the point in question by the distance from center to center of columns, or more accurately, from the centers of gravity of the combined column and portal section. If, then, the tensile or compressive stress in either leg of the portal is represented by S_t, the amount of the stress may be found by the formula

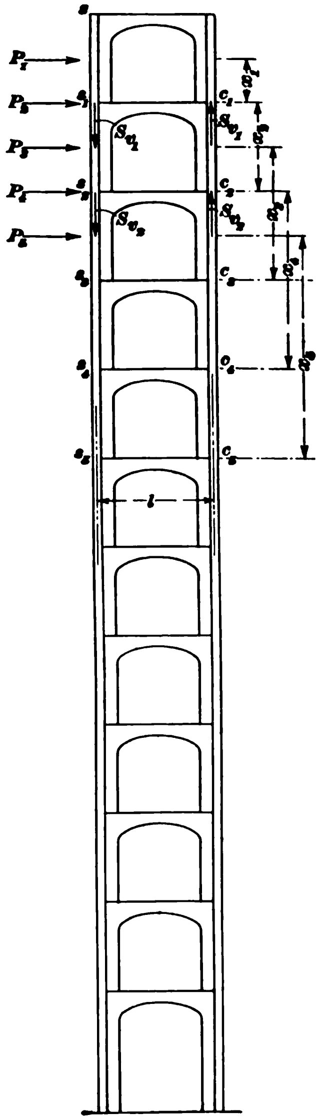


Fig. 31

$$S_t = \frac{Px}{l} \qquad (21)$$

in which

P = wind pressure on entire surface of wall above floor in which the columns are located;

x = perpendicular distance from center of effort, or the line of action of wind pressure to point of rotation around which structure tends to turn;

l = distance from center to center of columns, though more accurately could be regarded as distance between centers of gravity of combined sections of columns and portal legs.

The value obtained by this formula is also the shear on all vertical planes throughout the portal, while the horizontal shear in either leg of the portal throughout its length and at the bottom is equal to the horizontal

reaction due to the wind pressure P; its amount is expressed by $\frac{P}{2}$.

The thickness of the web-plate in the portal must be proportioned to resist the shearing stresses, while the connection of the web of the portal to the columns must offer a resistance equal to the vertical shear; there should also be sufficient bolts holding the bottom of the portal leg to the floor system to equal in resistance the amount of the horizontal shear, or $\frac{P}{2}$. The horizontal shear, therefore, throughout either leg of the portal at any floor is equal to one-half the resultant wind pressure on all of the surface above, or if, in Fig. 30, P equals the resultant,

$$S_h = \frac{P}{2} \qquad (22)$$

in which S_h = horizontal shear at the floor in question.

The vertical shear at any point is equal to the tension or compression in the windward or leeward columns and it is expressed by general formula **21**, or $S_s = \frac{Px}{l}$. From this formula and reference to Fig 31, it will be observed that the vertical shear throughout the portal at the second floor from the top is equal to

$$S_{v_2} = \frac{P_2 x_2}{l} - S_{v_1} \qquad (23)$$

in which P_2 = resultant wind pressure on all surface above point in question;

x_2 = perpendicular distance from point of application of load to point c_2.

Similarly, the vertical shear in the third floor from the top is equal to $S_{v_3} = \frac{P_3 x_3}{l} - S_{v_2}$. The values obtained from these formulas are the shears on all horizontal and vertical planes throughout the portal. From these shears the thickness of the web-plate in the portal must be proportioned,

and on them depends the design of the connection between the web of the portal with the columns and the floor girders; there should also be sufficient bolts holding the bottom of the portal leg to the floor system to equal in resistance the amount of the horizontal shear, or $\frac{P}{2}$.

The bending moment at the several points throughout the portal determines the dimensions of the flanges of the portal and the increased area that it is necessary to add to the columns or girders in order to provide sufficient resistance to this stress. There is no bending at the crown of the portal or the point w, Fig. 30, for it will be found that the algebraic sum of the moments about this point equals zero. The bending stresses should be calculated for several points throughout the portal. For instance, at the point u on the flange of the portal opening, the bending moment is equal to the vertical shear multiplied by the ratio between the two lever arms x' and y', or

$$M = \frac{S_v x'}{y'} \qquad (24)$$

The distance y' is the perpendicular distance between the tangent to the point u in question and the center of moments z, which lies at the intersection of the center of gravity of the flange of the girder and the perpendicular $z\,u$. The greatest bending moment is created in the flange of the portal at the point where the quotient of $\frac{x'}{y'}$ is the greatest.

21. Conservative engineers, in proportioning the flange of the portal opening, consider the leg of the portal as a cantilever subjected to bending by the horizontal force $S_h = \frac{P}{2}$, the vertical force $S_v = \frac{P\,x}{l}$ being disregarded, owing to the fact that this force is usually imposed along the axis of the columns or the center of gravity of the combined column and portal section. The leg of the portal considered in this way is subject to bending on the flange of the

portal opening at m, and the stress in the flange is found by the formula

$$S_f = \frac{S_v x''}{y''} \qquad (25)$$

so that, as before, the maximum ratio between x'' and y'' determines the location of the greatest bending moment.

In the method for determining the stress in the flange of the portal, the fact that the portal and columns are rigidly connected throughout has been disregarded. This condition tends to shorten the leverage of the force S_h, or the distance x'', so that by considering the legs free at the end, the flange will have an excessive area and the error that exists has the advantage of being on the side of safety.

22. When the portal has been properly proportioned to provide for the vertical and horizontal shears and the bending moments, attention should be directed to the construction of the portal legs, the floor girders, and the column splices, if they are affected. There is an overturning moment at each floor level, due to the pressure on the surface of the wall above by its lever arm about the foot of the leeward column, that is equal to Px. With reference to Fig. 31, this moment at the several floors may be expressed by the following:

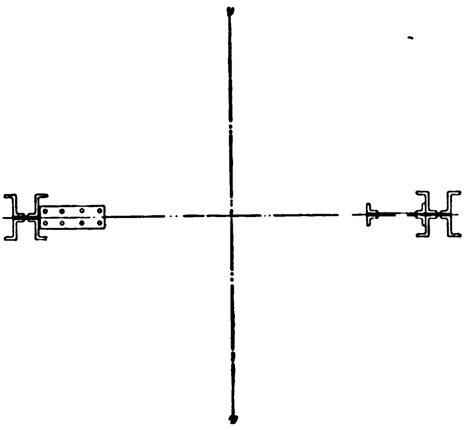


FIG. 32

$$\text{Moment about } c_1 = P_1 x_1$$
$$\text{Moment about } c_2 = P_2 x_2$$
$$\text{Moment about } c_3 = P_3 x_3$$

This overturning moment at each floor level is resisted on the compression side by the column and portal sections, and

on the tension side by the bolts or rivets securing the foot of the portal to the floorbeams, or girders, and the column sections, or in case of a column splice, by the shearing of the rivets in the splice.

23. For the resistance of this overturning moment, the section through any tier of columns including the portal bracing may be regarded as a beam section. The neutral axis of this section may be assumed along the center line of the portal system and the moment of inertia of the column sections, and the connections of the portal to the floorbeam or girder may be calculated. This assumption will not be entirely correct, for the resisting section on the windward side of the portal is equal to the column section and the tensile resistance of the bolts connecting the foot of the portal to the floorbeams, while on the compression side of the portal, the resisting section consists of the section of the portal and the section of the column, so that the entire section is not symmetrical along the assumed neutral axis and the neutral axis will be moved nearer to the compressive side of the portal. Since, however, the opposite sides of a building are subjected alternately to the wind, it is reasonable to consider the neutral axis as the center line of the portal section. Therefore, in order to determine the resisting moment of the columns and the portal connections to the floorbeams, it will be necessary to calculate the moment of inertia I about the neutral axis yy, Fig. 32. The elementary sections on the one side will be considered as including the column section and the section through the bolts or rivets securing the foot of the portal to the floorbeams or girders, while about the other side of the neutral axis the elementary sections will be taken as a section through the portal and through the column. The section on which the calculations will ordinarily be made is shown in Fig. 32. In this figure, the material to the left of the neutral axis is subjected to tension while the material to the right of the neutral axis is subjected to compression. The method for calculating the moment of inertia I of any section is described

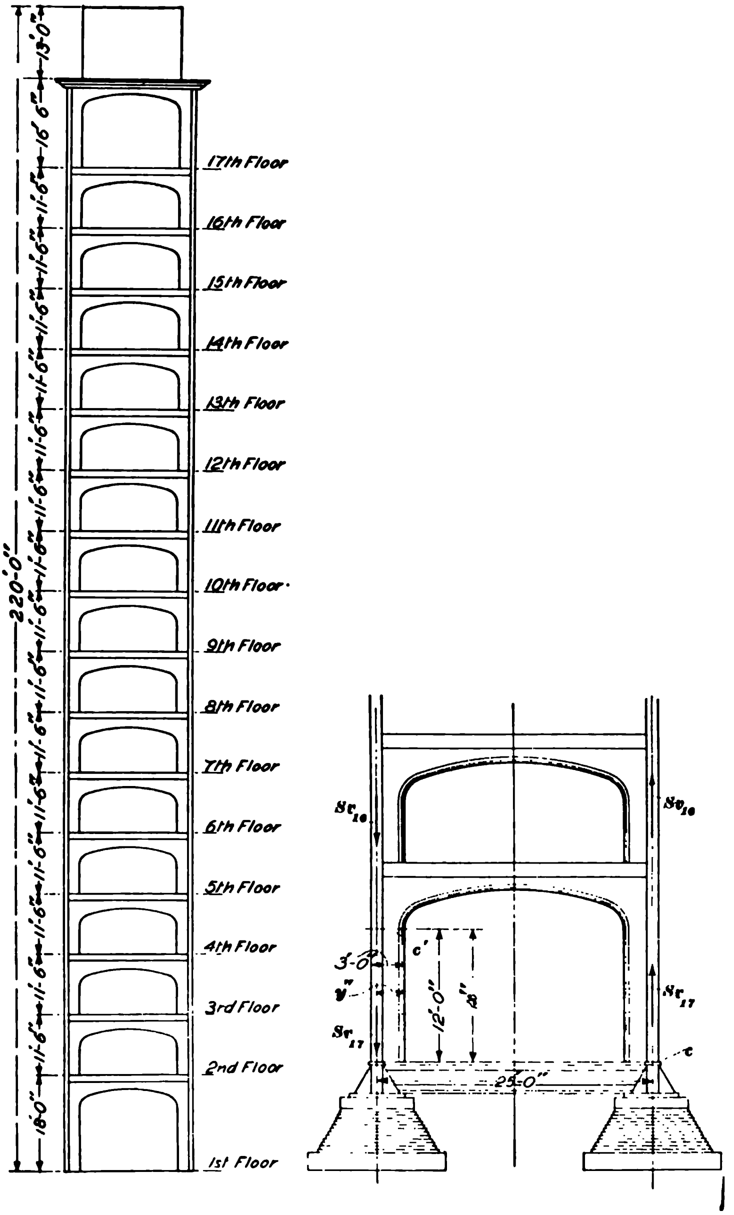

17th Floor
16th Floor
15th Floor
14th Floor
13th Floor
12th Floor
11th Floor
10th Floor
9th Floor
8th Floor
7th Floor
6th Floor
5th Floor
4th Floor
3rd Floor
2nd Floor
1st Floor
220'-0"
13'-0"
16'-6"
11'-6"
18'-0"
3'-0"
12'-0"
8"
25'-0"
c'
c

FIG. 33

under the heading Moment of Inertia, in *Properties of Sections.* Since the bending moment must equal the resisting moment $M = M_1$, and as $M = Px_1$ and $M_1 = s\frac{I}{c}$, $Px = s\frac{I}{c}$, when the unit fiber stress sustained on each square inch of rolled section and bolts must equal $\frac{Pxc}{I}$, in which c equals the distance from the neutral axis, or the center line of the portal, to the extreme fiber of the column section on either side. This equation is simply the application of the formula for the resistance of any beam, given under the heading Resisting Moment in *Properties of Sections.*

EXAMPLE.—In order that the portal bracing of the seventeen-story building shown in Fig. 33 may be designed, it is necessary to determine the several stresses at the bottom tier of floorbeams. The height of the area exposed to the wind, assumed to come from the left, is 220 feet and the width of the area supported by one portal is 20 feet. Provided that a maximum wind pressure of 20 pounds per square foot is assumed, what will be: (*a*) the maximum tension or compression in the column or leg of the portal? (*b*) the maximum stress in the inner flange of the portal? (*c*) the maximum shear on the web-plate of the portal?

SOLUTION.—(*a*) The area exposed to the wind is $220 \times 20 = 4{,}400$ sq. ft., and the total pressure on this surface is $4{,}400 \times 20 = 88{,}000$ lb. The moment about the point c due to the resultant of the wind pressure is, consequently, $Px_{17} = 88{,}000 \times 110 = 9{,}680{,}000$ ft.-lb.

In the previous expression x_{17} is merely the symbol for the leverage of the wind effort acting about the first story. If it had been the purpose to express the leverage of the wind about the next story above, the symbol x_{16} would, for convenience, have been used. Likewise, further along in the solution the values $S_{v_{16}}$ and $S_{v_{17}}$ occur; these are simply used to designate the vertical shear in the first and second stories above the foundation. The value l, which in this case is considered as the distance from center to center of columns, though it would involve greater nicety to consider it as the distance from the centers of gravity of the combined sections of the portal and columns, is equal to 25 feet, and the tensile stress in the windward column and the compressive stress in the leeward column from formula **21**, or $S_s = \frac{Px_{17}}{l}$, is

$$9{,}680{,}000 \div 25 = 387{,}200 \text{ lb. Ans.}$$

It must be remembered that while 387,200 lb. is the gross theoretical tensile stress in the windward column, the overturning tendency is partially, if not entirely, resisted by the weight of the building acting

in opposition to the overturning moment of the wind about the point c. In this case, the leverage of this great weight will be 12 ft. 6 in., and it will be found that the stability of the building against overturning will probably more than equal the moment of the wind pressure, so that no provision need be made for the resistance of the tensile stress in the windward column, but it would be conservative practice to make an allowance for the compressive stress in the leeward column and the amount of compressive stress due to the wind, 387,200 lb., should be added to the dead and live loads on this column.

(b) The maximum stress in the inner flange of the portal, due to the horizontal reaction at the foot of the portal equal to half the resultant pressure of the wind, or $\frac{P}{2}$, occurs at the point c', and the maximum bending moment at this point equals $\frac{Px''}{2}$; the tensile or compressive stress, as the case may be, in the flange at c' is

$$\frac{Px''}{2y''} = \frac{44{,}000 \times 12}{2 \times 3} = 88{,}000 \text{ lb. Ans.}$$

To provide for a stress of 88,000 lb. sufficient flange area to resist either the compression or tension must be provided at the point c', though a portion of the web-plate can be considered as forming some of the flange area.

(c) The horizontal shear on the web of the portal is equal to $\frac{P}{2}$, or 44,000 lb. The vertical shear on the portal is found by formula **23** to equal $S_{v_{17}}$, or $\frac{Px_{17}}{l} - S_{v_{18}}$. The first part of the expression, $\frac{Px_{17}}{l}$, equals $\frac{88{,}000 \times 110}{25} = 387{,}200$ lb., and the amount to be deducted from this, or $S_{v_{18}}$, the stress on the column above, is equal to 326,432 lb., so that the vertical shear $S_{v_{17}}$ equals

$$387{,}200 - 326{,}432 = 60{,}768 \text{ lb. Ans.}$$

This last value obtained is the maximum shear on the web of the portal, but owing to the fact that the width of the web is greater at the crown of the arch of the portal than at the sides, it is possible that the horizontal shearing stress creates a maximum unit shear and that it will be necessary to proportion the thickness of the web for the horizontal shear instead of the vertical. The method of procedure in proportioning the web of the portal is the same as that employed in proportioning the web for plate girders described in *Beams and Girders*.

EXAMPLES FOR PRACTICE

1. Referring to Fig. 33, determine the horizontal reactions at the foot of the portal: (a) on the seventh floor; (b) on the fourth floor.

Ans. { Seventh floor, 28,900 lb.
Fourth floor, 35,800 lb.

2. What will be the maximum vertical shear on the web of the portal bracing shown in Fig. 33 at the fifth and tenth stories?

Ans. { Fifth story, 19,182 lb.
Tenth story, 29,562 lb.

3. Determine the maximum bending moment on the columns and portals due to the horizontal reaction at the foot of the second and eighth stories, Fig. 33.

Ans. { Second story, 10,665,600 in.-lb.
Eighth story, 42,453,600 in.-lb.

4. Determine the tensile and compressive stresses, due to the wind pressure, in the windward and leeward columns of the portal bracing at the sixth story, Fig. 33. Ans. 194,888 lb.

DETAILS OF DESIGN

24. The columns of a building of the skeleton-construction type, wherever possible, should be continuous from the foundation to the roof. Where splices occur, it is best, if the design will admit it, to butt the ends of the columns that have been milled and provide splice plates extending from 2 to 3 feet each side of the joint and on all four sides of the column. This type of splice for structural steel columns, shown in elevation and plan in Fig. 34 (*a*), makes a much stiffer connection than the one shown in (*b*), where the column above is secured to the one below through a plinth plate. With a carefully designed spliced connection a continuous column from foundation to roof is practically obtained, and the best practice in the design of tall buildings requires that the splices shall only occur at every other floor and that the joints shall be broken so that alternate columns are spliced at different floors, as shown in Fig. 35.

The plinth-plate connection shown in view (*b*) of Fig. 34 is now seldom used, from the fact that though it offers considerable resistance to the horizontal shear and sufficient to the upward tensile stress created in the windward column by the wind pressure, which is rarely great from the fact that it is counteracted by the dead loads, it offers little resistance to bending. The continuity desired in the columns may be obtained by the employment of the splice shown in Fig. 34 (*a*), and such a splice is absolutely necessary in

buildings where adequate wind bracing is not provided, and where the lateral stability of the structure depends on the usual beam-and-girder connection. The splice connection for structural steel columns should be used in all systems of wind bracing where a bending moment is created in the columns, whether they are knee- or portal-braced. The diagonal, or sway-brace, system does not exert any bending stresses on the columns; consequently, the consideration of continuity of the column from foundation to roof is not of vital importance, so that where economy demands it and the facility of erection requires it the plinth connection shown in Fig. 34 (*b*) may be used.

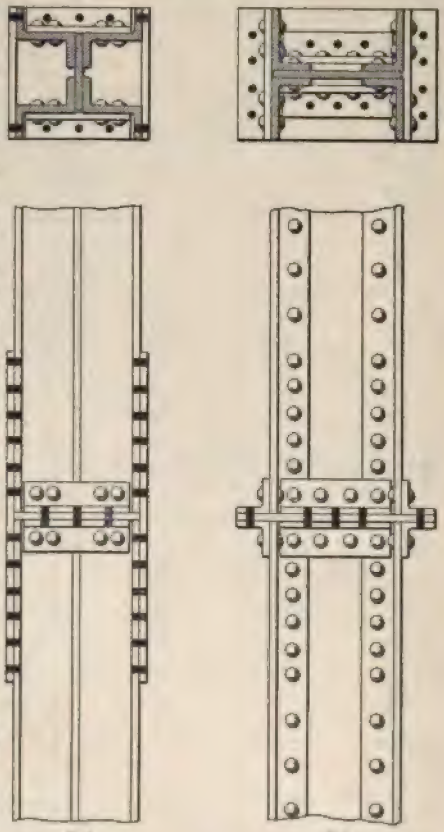


Fig. 34

25. The details for gusset- or angle-plate bracing usually employed are shown in Fig. 36. Views (*a*), (*b*), and (*c*) show two types of gusset-plate wind bracing; (*a*) shows a channel or latticed column supporting a floor principal consisting of a box section *a*, made of two channels placed back to back at the same distance apart as the channels in the column and held together with binder plates, as at *g*. The gusset-angle bracing *b* is composed of two triangular plates riveted to the back of the channels forming

the column and the floor principal. These gusset plates are reenforced by angle irons and the knee braces thus formed are secured by rivets through the flanges and webs of the channels, forming the columns and the floor principals. It is good practice in such a construction to carry the lattice bracing, or to provide tie-plates, as at *h*, along the oblique reenforcing angle of the gusset plate, since it is necessary to omit the lattice bracing of the column, where the angle or gusset-plate bracing occurs. The connection of the lower column with the upper is made in this detail by means of the splice plates *d* and *e* and by means of the plinth connection at *f*.

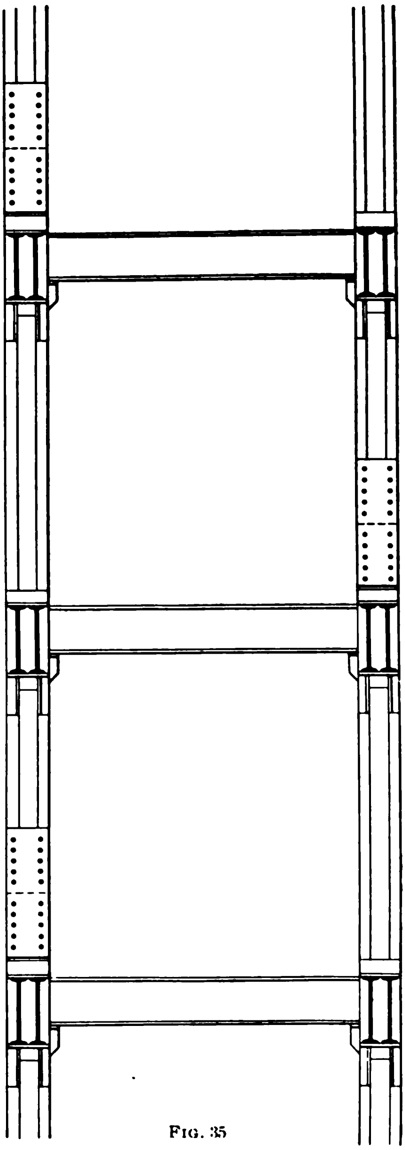


FIG. 35

View (*b*) shows the gusset-plate bracing in connection with the plate-girder floor principal. The column is an outside one and the curtain wall is supported on the double I-beam girder shown at *i*. In this gusset-plate brace, the reenforcing angles along the oblique edge have the flanges turned inwards, which facilitates riveting somewhat. The splice

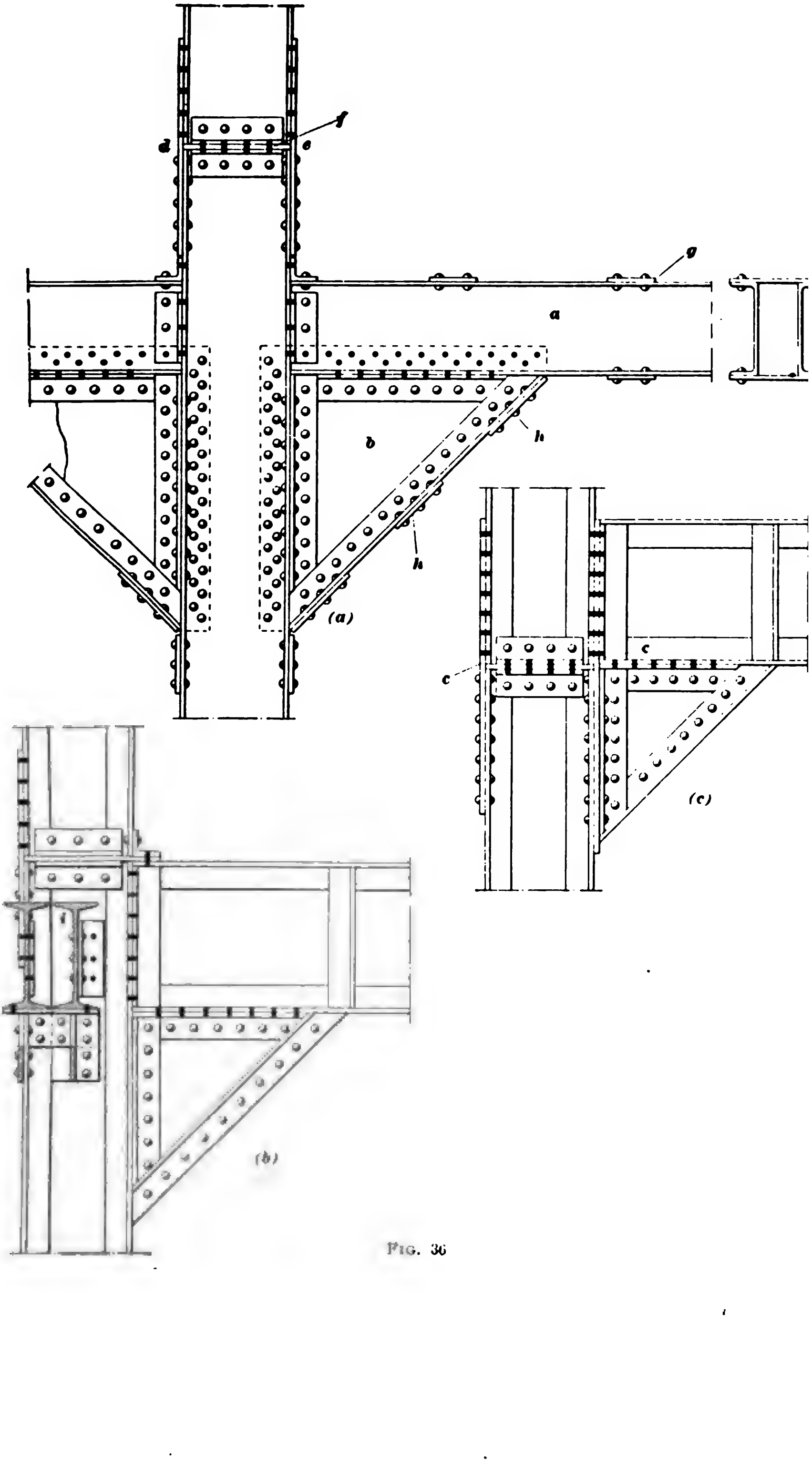


FIG. 36

connection of the column is conveniently made at the top flange of the floor principal and is a combination of the splice-plate and plinth connection. In this construction the plate-girder connection to the column assists considerably in resisting any lateral movement, and in fact where the floor principals or plate girders are of considerable depth, the gusset brace, especially in buildings from twelve to fifteen stories,

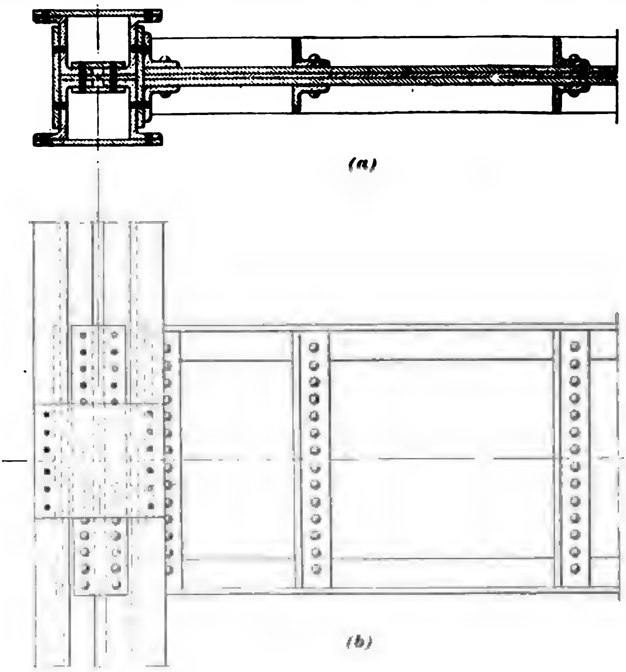


FIG. 37

is neglected, and the connection shown in Fig. 37 is employed. In this figure, view (*a*) is an elevation and (*b*) a sectional plan.

For buildings of considerable area, where there are several rows of columns, it is not unusual to use the gusset brace at the junction of the floor principals and the exterior columns and the row of columns adjacent thereto, and to use a modification of the detail in Fig. 37 for the interior columns. In Fig. 36 (*c*) is shown the same type of gusset brace

applied between a plate girder and column of similar construction. The splice of the column, however, is laid on a level with the lower flange of the girder and the girder is connected to the bottom of the upper column, while the gusset brace is connected to the lower. This can hardly be considered as rigid and secure a connection as the one shown in view (*b*), though there is an advantage in the fact that two splice plates *c*, *c* may be provided instead of one, as in (*b*). However, it is preferable, where possible, to secure the girder and the gusset brace to the same column rather than to the upper and lower columns, as shown in this detail. The plinth plate, in this figure, connecting the upper and lower columns can be used to assist in supporting the outside girder carrying the curtain wall. The details shown in (*b*) and (*c*) would be likely to be used at the first tier of beams carrying the curtain wall, the wall beneath being independent and self-supporting from the foundation. In the economical construction of a building twelve or fifteen stories in height, lateral movement in the columns and frames for all floors beneath the floor carrying the first portion of the curtain wall will be resisted by the rigidity of the beam connections of the floor systems.

26. In Fig. 38 is shown a type of double gusset bracing; the connection is made still more rigid by splicing the column at the center of the plate girder. This connection is used for tall and narrow buildings when the sway brace or portal is not applicable. In all these connections the greatest rigidity is obtained by riveting instead of bolting, and the rivets, where possible, should be hot-driven with a machine, so as to be sufficiently upset to fill the hole neatly, and to make the entire connection rigid. Failure can then occur only by the shearing of the rivets. In designing any gusset connections, the stresses may be calculated by the formulas given for gusset plates and knee braces under the heading Determination of Stresses for Bracing High Structures. In applying these formulas, a skeleton diagram should be drawn and dimensioned; the diagram should coincide with

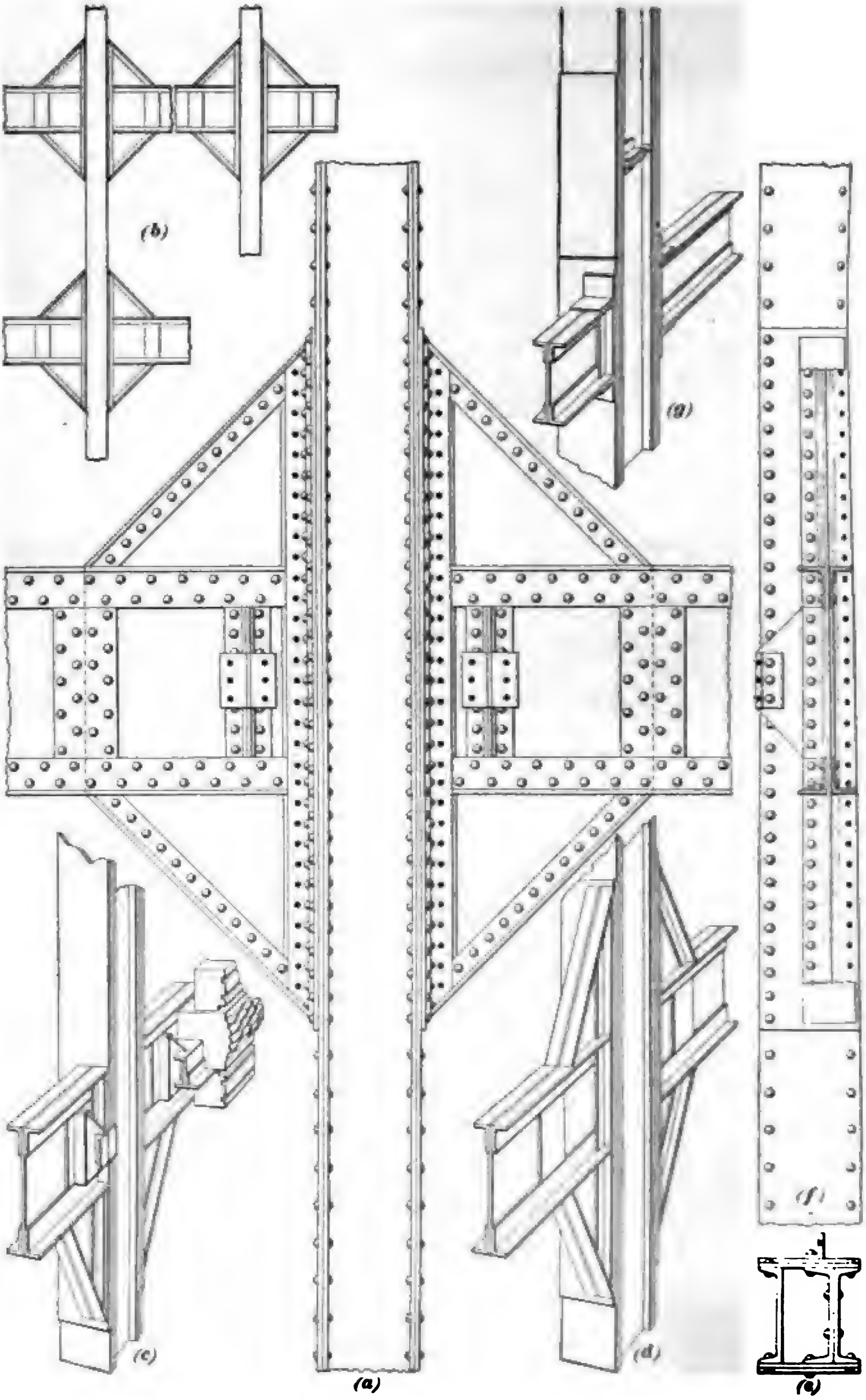


FIG. 38

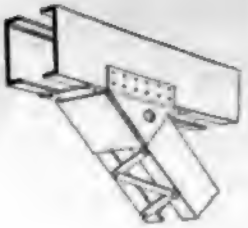
the axis of the column, a line passing through the neutral axis of the oblique angles reenforcing the gusset plate and the center line or neutral axis of the plate girders or floor principals.

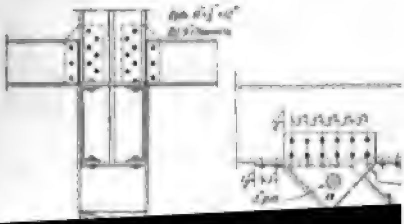
27. Some usual details of knee-bracing are shown in Fig. 39. At (*a*) is shown a system in which both the horizontal and diagonal members are composed of channels securely tied together at their ends with plates and in some instances latticed throughout their length. Where a pin connection is used in this way excellent workmanship should be insisted on and the pin should approach a driving fit so that there will be no motion in the frame. The horizontal member may be either independent of the floor system or be used as one of the floor principals.

Fig. 39 (*b*) shows the dimensioned details for the knee-braced frame shown in view (*a*). It will be noticed that there are many field rivets to be driven. Some of the rivets that are blackened, thus indicating field rivets, could be driven in the shop, but to avoid the danger of bending such plates as *a*, *a* during shipment and erection, it is advisable to rivet them in the field.

The advantage existing in a pin-connected knee brace is that the stress on the member is transmitted along the neutral axis. A pin-connected knee brace has an advantage over one with riveted ends, from the fact that the accurate determination of the resistance of the knee brace when acting as a column or strut with fixed ends is difficult. The detail design of the wind brace greatly depends on the height of the column employed and the construction of both the column and horizontal member. The principal consideration in designing knee-braced frames is to observe that good connections can be made at the ends of the knee braces to the horizontal and vertical members.

28. Two types of knee braces that are usually employed in connection with roof trusses are shown in Fig. 40 (*a*) and (*b*). In (*a*) is shown a knee brace composed of two angles, back to back, rigidly secured to the latticed girder at *a* and to the angle column at *b*. Such a brace may





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ASTOR, LENOX AND
TILDEN FOUNDATIONS

be proportioned by formula **1**, or by the method described in *Graphical Analysis of Stresses*, Part 2.

In (*b*) is illustrated a pin-connected knee brace incorporated in the design of the roof-truss connection with its supporting column. This detail is in its purpose very similar to the one shown in Fig. 40 (*a*). The channel bars composing the brace, shown in detail at (*c*), can be secured together so as to act in unison and provide a greater resistance to compression either by latticing, as shown, or by the use of tie-plates. In any case, one of these tie-plates, as at *d*, *d* in (*c*), should be placed as near as possible to each pin connection and, if intermediate tie-plates are used, the distance from center to center of such plates throughout the length of the member should not be greater than four times the least dimension of the strut. Owing to the bends in the two channels of the strut shown, the best construction is to employ the lattice tie.

From an examination of the detail, it will be observed that the channels adjacent to the pinholes are thoroughly reenforced by the pads or plates riveted to their webs. This is necessary to secure greater bearing resistance in the metal around the pinhole and to provide additional security against the tearing of the metal. The pads or plates shown in the figure at (*c*) also reenforce the thin web of the channel against bending sidewise, and are especially useful when the upper and lower flanges have been cut away to facilitate the assembling of the connection, though that has not been necessary in this instance.

The sides of the frame shown in Fig. 40 (*b*) are covered with corrugated iron, which is secured by means of clips or is riveted directly to a light framework, composed of **Z** bars and angles. This framework is shown in detail in (*d*), in which *e* is a plan and *f* an elevation of the **Z** bars shown at *c* in (*b*) together with the angle connections.

29. In the construction shown in Fig. 41 it will be noticed that the knee braces are composed of two channels placed back to back. They are connected at the ends to the

heavy floor principal *a* and the columns *b*, *b* by means of field-driven rivets. The web-plate of this principal is made in several pieces, one of which, *c*, extends below the lower flange. In order to provide stiffness in the web-plate of the girder when it has been built in several pieces in this manner, it is necessary to furnish the reenforcing plates shown at *d*, *d*, the double row of rivets at either end being required to form the necessary splice between the two portions of the web-plate. The lower end of the knee brace is connected to a gusset secured to the column by filler

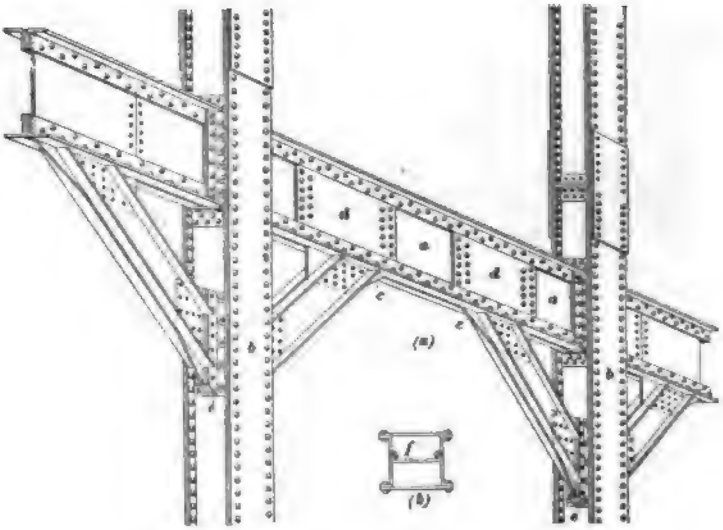
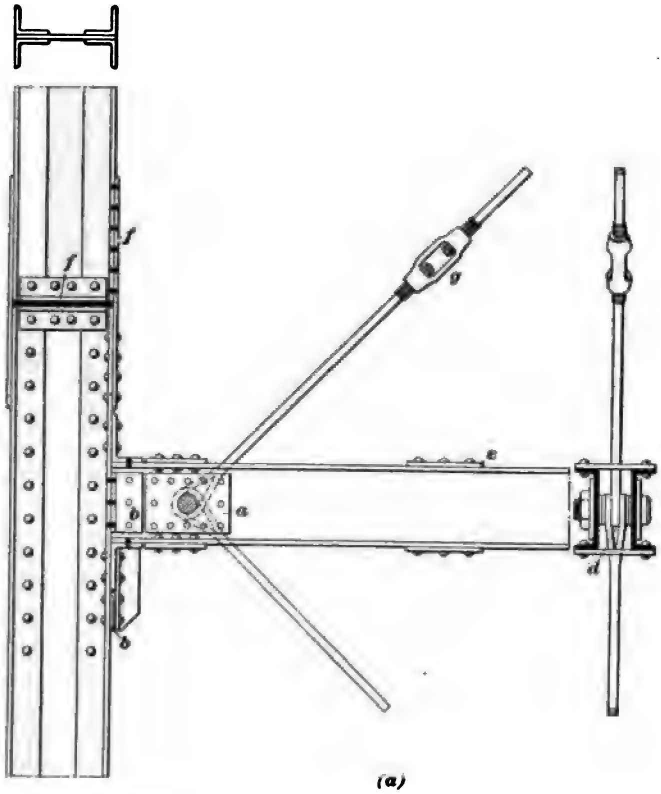
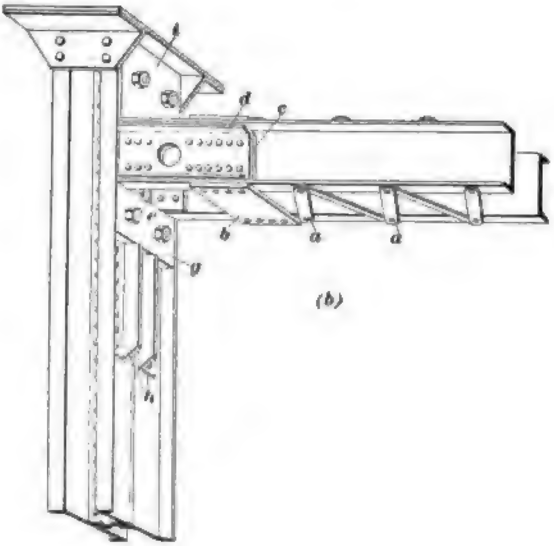


FIG. 41

plates, angle clips, and through bolts *e*, *e*. The weak point in this design is due to the fact that some of the thrust of the knee braces will be exerted on the web of the channels forming the column, so that the column should be reenforced at these points by a diaphragm plate, or channel *f*, as shown in the section of the column. This diaphragm plate will have to be held in place by rivets countersunk on the outside. By the use of heavy channels for the column section, however, the ordinary wind brace necessary for a frame of moderate height will not require such a diaphragm reenforcement.



(a)



(b)

FIG. 42

The other details in the figure are sufficiently clear, so that further explanation is unnecessary.

30. In Fig. 42 (*a*) and (*b*) are shown details of construction of sway-rod, or diagonal, braced frames. An analysis of the stresses at this connection shows that the forces at the junction of the horizontal struts and columns are peculiar, as they consist of an oblique stress due to the tension in the sway-rod, or diagonal, brace. This stress is the resultant of a horizontal stress that is resisted by the compressive stress of the wind strut, or horizontal member, and the vertical stress due to the weight of a floor system, if the wind strut also acts as a floor principal.

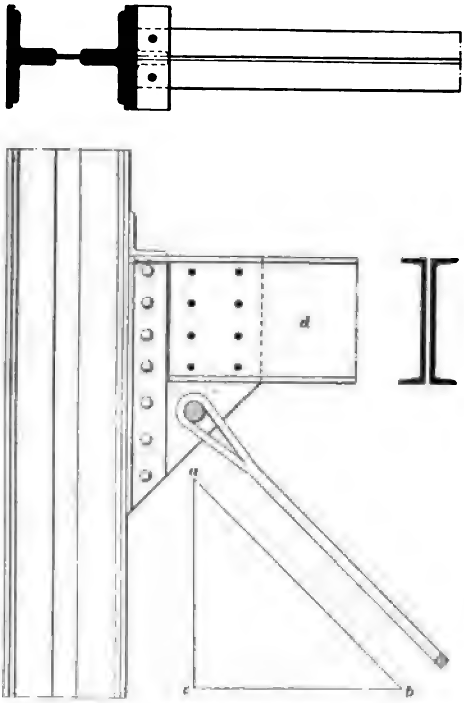


FIG. 43

The oblique force on the pin from the sway-rod may be resolved into the horizontal and vertical components by applying the principles explained under the heading Resolution of Forces, in *Graphical Analysis of Stresses*, Part 1. For instance, assume that it is desired to obtain the vertical stress due to the pull in the oblique member shown in Fig. 43. According to this figure, the oblique stress is 20,000 pounds; then by laying off this amount, to scale, on a line $a\ b$ extending in the direction of the oblique member and drawing $b\ c$ and $a\ c$ parallel,

respectively, to the horizontal member or wind strut *d* and the vertical member, or column, the amount of the vertical component in the oblique member may be obtained by measuring the length of the line *a c* with the same unit measurement employed in laying off *a b*. The amount of the vertical component of the oblique stress is equal to the increment of compressive stress created in the columns by the oblique members, which is shown in the notation given and described in Art. **17**. This vertical stress is the principal one to be provided for in the connection of the wind strut with the columns, and since it may act either upwards or downwards the connection must be designed for an equal resistance in either direction.

In Fig. 42 (*a*) is shown the most usual form of details for the sway-rod, or diagonal, wind bracing. Here, as in most details of this type of bracing, the sway rods are connected with the wind strut by a pin connection. The wind strut must be reenforced with a pad or plate *a*, for reasons previously explained, and the pin must be proportioned for shearing and bending by the methods described in *Details of Construction*.

There is no tendency in any system of diagonal or sway bracing for the ends of the wind strut to separate from the column, so that, theoretically, the end of the wind strut need not be connected with the column, provided it is supported above and below in a sufficient manner to resist the vertical component of the stress in the oblique member. The rivets or bolts *b*, *b* through the brackets or clips, Fig. 42 (*a*), are simply for the purpose of resisting a vertical stress and are consequently subjected only to vertical shear. In determining the number of these rivets, where the wind strut is also a floor principal, the vertical shear due to the vertical component from the oblique member must be added to the amount of the vertical reaction from the floor loads at the end of the wind strut. By observing this detail carefully, it will be noticed that the connection will have considerably more resistance in a vertical direction downwards than upwards, and it must be remembered, in designing the connection for

the upward force due t
in the oblique member
opposite side of the b
is counteracted by the
system; therefore, the
algebraic sum of the
principal and the ver
oblique member. For
also acts as the floor
from the weight of t
pounds; also, that the v
stress in the oblique m
to 30,000 pounds; rega
the floor as negative a
wind pressure as posi
tical stress of – 10,00
vertical component du
and that there is no u
must be observed, how
also as a floor principal
loads due to the weigh
provided for in design
strut, for it is seldom
realized, and usually th
will be inappreciable.

31. Another featur
is the clevis connectio
pin, as shown at *d*. It
tion is necessary in orde
in the same vertical pla
wind strut or horizonta
tied together at interval
lower flanges. The co
the usual plinth-plate,
nection *f, f*. It is cu
members with turnbuckl
that they are accessible

r, the turnbuckle *g* is in such a position as to be not e than 3 or 4 feet above the horizontal member. en placed in this position, it can always be reached from folding resting on the member.

nother type of sway-rod, or diagonal, wind-brace con- ction is shown in Fig. 42 (*b*). This is an unusual il, but it is one that has been used by prominent engi- rs in the construction of large narrow office buildings. wind strut is composed of two 9-inch channels secured ether by latticed bars *a* and connecting plates *b*; the chan- bars are reenforced adjacent to the pinhole by reenforcing es *c* and a reenforcing 8-inch channel *d*. A peculiarity be noticed in that there is but a comparatively insecure nection between the wind strut and the column, and igh this connection is slight, the strut must fit exactly veen the two columns to which it is secured. With this in view, the wind strut is made accurately and is planed to length at both ends, no clearance being allowed veen the ends of the strut and the columns that it abuts. open holes *e* are provided for bolts to hold the wind strut lace and to facilitate the erection of the frame. Beneath wind strut, there is provided a solid cast-iron block *g* ported on two bracket angles *h*. Sufficient rivets are pro- ed in the legs of these angles to resist the vertical com- ents of the stress in the sway rods. Above the end of wind strut there is provided a cast-iron block *i* planed on top and bottom so as to fit tightly between the strut the cap plate of the column. This block is made to fit the ss formed by the flanges of the **Z** bars so closely that the ch cap plate is brought into direct shear entirely around e sides of the block. In this way there is provided suffi- t resistance to any upward pull from the vertical compo- t in the diagonals when the wind action is reversed. The gners considered that the shear resistance of this plate, ether with the weight of the beam directly on it, will be e than enough to resist the upper vertical components of rods. The cast-iron block *g* is of sufficient depth to resist transverse stress that may be created, from the fact that

the bracket angles *h* are not directly under the bearing of the channels forming the wind strut.

It is never good practice to place the pinholes as far from the column as is shown in Fig. 42 (*b*). However, it is sometimes necessary to give the diagonal sway bracing a sharper slant, in order to clear door or window openings; this condition influenced the design shown in the figure. When

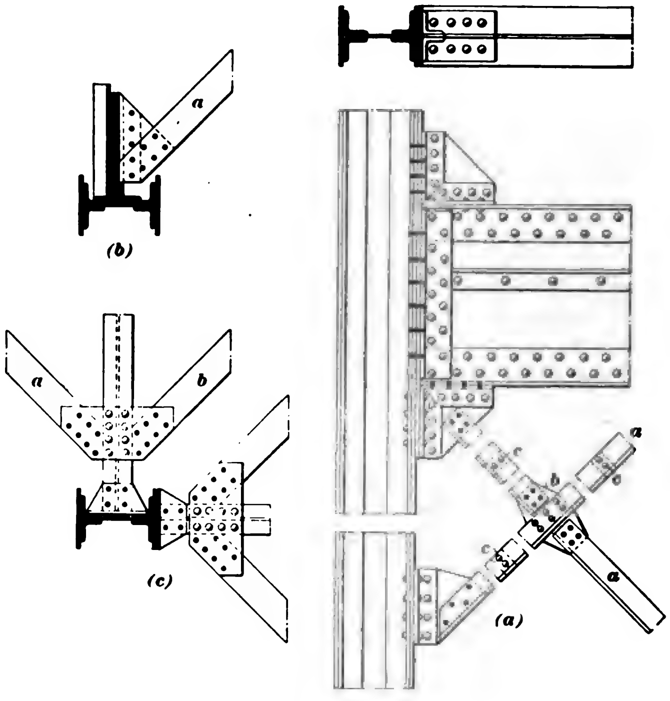


FIG. 44

pinholes are placed such a distance from the column, the wind strut must be heavily reenforced adjacent to the pinhole, in order to provide sufficient resistance for the transverse stress created by the eccentricity of the pin connection.

32. Frequently, sway rod, or diagonal bracing, is made with riveted connections, and of rolled shapes, as shown in Fig. 44 (*a*), (*b*), and (*c*). In (*a*) is shown an angle web-plate

and cover-plate column to which is securely attached a floor principal consisting of a plate girder. This member answers the double purpose of a floor support and wind strut. The diagonal system of bracing is composed of heavy angles *a*, *a* properly secured to gusset plates riveted through angle clips to the column and wind strut. Where these diagonal members intersect, a splice connection *b* is provided. In this instance, the diagonal brace is composed of two angles back to back, and the necessary separators *c*, *c* must be riveted at intervals throughout their length.

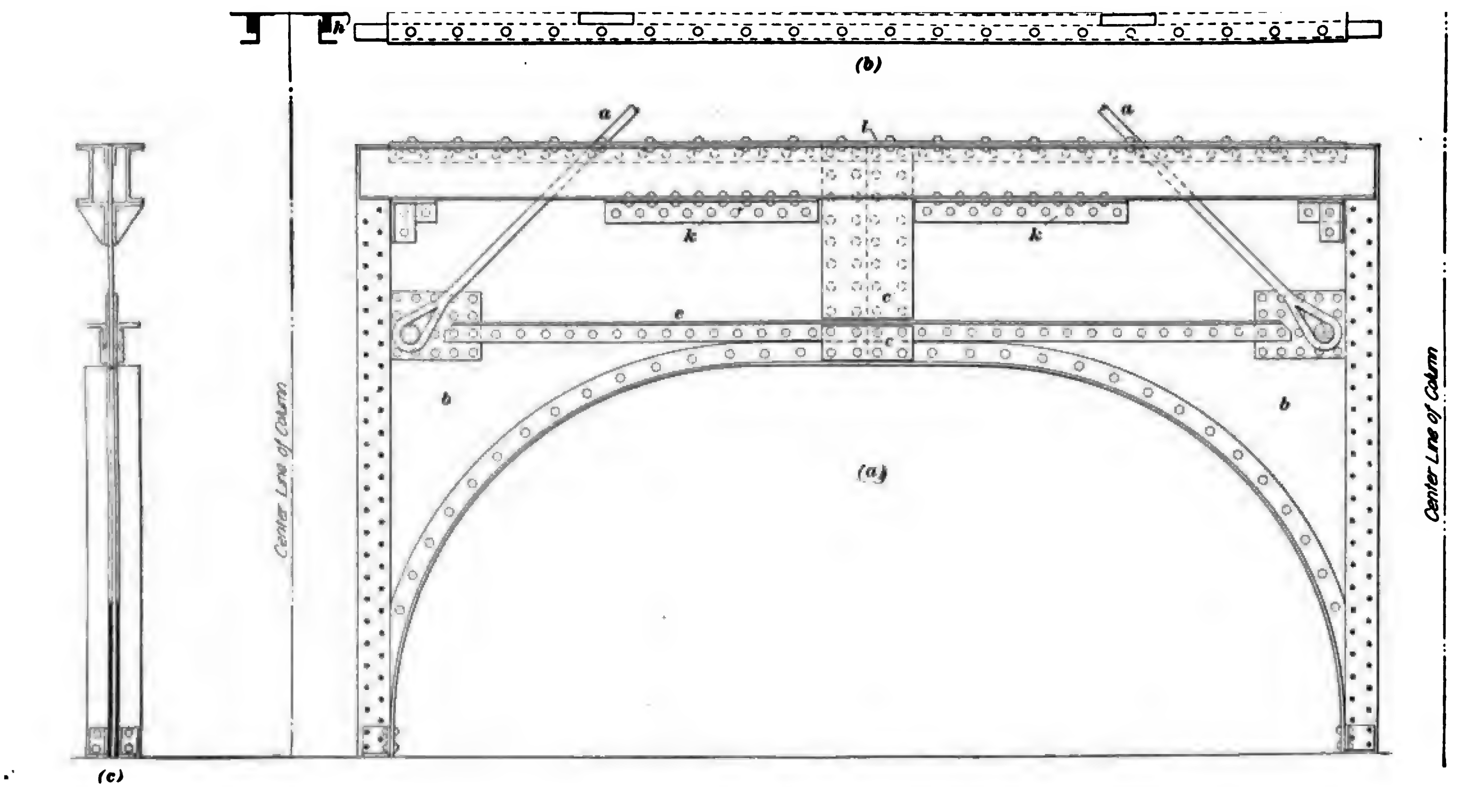
Where the lateral rigidity of the floor construction is in question, horizontal wind bracing is sometimes provided between the columns, as shown in views (*b*) and (*c*). In both these figures the diagonal bracing consists of $5'' \times \frac{1}{2}''$ pieces of bar iron *a*, *a* that are riveted securely to gusset plates, which, in turn, are fastened to the members of the floor system.

33. Were it possible, the ideal construction of the sway-rod connection would be to have the pin located at the intersection of the axes of the wind strut and the column. This, with the usual structural column and wind strut, can rarely be accomplished. It has been stated that with the sway-rod, or diagonal, bracing there is no bending moment created. This is only true of a theoretical construction; that is, where the axes of the wind strut, column, and the diagonal tension member intersect at a single point. Where the tension or sway rods are secured to the wind strut at some distance from the column, as shown in the details in Fig. 42, there is considerable bending stress created in the wind strut, and this transverse stress is equivalent to the bending moment created by a force located at the center of the pin connection, and is equal to the vertical component of the stress in the oblique member. The wind strut, as usually shown, however, will be found to provide sufficient resistance for any bending moment created by the vertical component of the stress in the oblique member. If the sizes, as originally assumed, do not provide for resisting such a bending moment, the section of the wind strut should be

increased. When the wind strut forms one of the floor principals, it is necessary to consider this force as tending to create bending, together with the transverse stress due to the loads on the auxiliary floorbeams and the maximum bending moment ascertained as described in *Beams and Girders*, Part 1.

34. In Fig. 45 is shown the detail design of the portal brace. The columns are **Z** bars on sections heavily reenforced by web-plates and angles riveted to the web of the **Z** bars. The portal brace shown in elevation in (*a*) and (*c*) and in plan in (*b*), is a type of wind bracing that was used in the basement story of a large office building. The upper panels of the frame are braced by means of sway rods *a*, *a*. The portal brace consists of two large web-plates *b*, *b* jointed together at the center by the splice *c*, *c*. The portal is reenforced by two 10-inch 30-pound channels connected to the web-plate at the top and bottom by 6″ × 4″ angles, forming brackets, as at *k* and *l* in the elevation. In order that the diagonal brace rods may pass through the upper member of the portal, it is necessary to slot the angles on the top, as shown at *d*, *d*. The horizontal thrust of the diagonal tension bars *a*, *a* is resisted by the web of the portal assisted by the reenforcing angles *e*. In order that the portal may make a neat finish in the interior of the building, it is reenforced by angles bent to form an arch. It will be noted that these portals do not extend to the foot of the columns to which they are attached; consequently, these columns, which are the basement columns of the building, must be very heavily constructed in order to resist the transverse stress created by the lateral wind pressure acting on the entire exposed surface of the building.

Another method of constructing a portal brace is to use **Z**-bar columns reenforced on the sides with a cover-plate. It will be noticed from the figure that the entire portal is secured to the columns by extending the web-plate marked *g* in (*b*) between the outside web-plates of the column section, the middle web-plate being stopped short and the portal


(b)
h
a
a
l
k
k
c
c
e
b
b
(a)
Center Line of Column
Center Line of Column
(c)
FIG. 45

being rigidly secured by means of field rivets. It is necessary, where the channels on the portal brace extend beyond the top angles connecting them to the web-plate of the portal, to pack between the web-plates of the columns and the channels with the proper packing plates, and attention is called to the fact that the two extended web-plates *h*, *h* are only used for the height necessary to secure the portal to the column. Where the necessity for these extended plates does not exist, the roof of the plate is reduced to correspond with the width of the central web-plate *i*. Where the web-plate is butted to the narrow plates beneath, proper reenforced splice plates are provided.

Another method of constructing portal bracing is to use Z-bar column sections reenforced on the sides with cover-plates. A single web-plate is extended so that the portal web may be connected to it by means of splice plates. Frequently the beams reenforcing the top of the portal are only auxiliary beams in the floor system and therefore carry a small proportion of the floor load. The portals are sometimes constructed in the shop in two pieces, separate from the columns. In this way shipment and erection are facilitated, the portals being connected to the columns and put together during erection by means of field rivets. While it is usually possible to secure web-plates of sufficient size to construct the portal without splicing, it is not economical construction, for all of the material included between a straight line drawn from the heel of the intrados and the crown of the portal and lying between the curved outline of portal is practically wasted, as the mills will not furnish sketch plates in which there is a concave cut without considerable expense for cutting and a charge for the waste material. It is therefore usual to splice the web of a portal brace at several points so that the cutting necessary to shape the plates to the required curve is reduced to a minimum. The columns in the portal type of bracing are usually more conveniently built and erected when they extend through one story only and the connection between column and column is frequently made on a line with the top of the

portal. In connecting the columns of portal bracing, the plinth-plate connection is usually employed.

35. Wherever splices in the web of a portal occur, sufficient rivets must be placed through the splice and web to sustain the vertical and horizontal shear at the point where such splices occur. If the flange angles around the portal opening are spliced, care must be observed in the design of the splice to provide that the strength of the net section of the metal in the flange angles is realized. The application of the formulas for determining the stresses in the different systems of wind bracing, and in the practical design of the details and the necessary calculations in conjunction therewith should be worked out from beginning to completion in original examples, which should be made as practical as possible and in which the best principles of modern design in connection with the wind bracing of tall buildings should be embodied.

SPECIFICATIONS

REMARKS ON SPECIFICATION WRITING

1. The most desirable quality a specification should possess is clearness of expression, and to attain this there must be in the mind of the writer a distinct image of the thing to be described or the process to be carried out. In brief, a specification that will satisfactorily accomplish its purpose cannot be written by one who does not thoroughly understand and cannot describe or explain, in exact language, the work to be executed.

The English language is capable of expressing almost everything of which a clear mental image can be formed; especially things so practical as descriptions of building construction. Therefore, if a thing or process can be distinctly thought of, it can be distinctly expressed, if the proper use of words is understood.

Words stand for ideas. Some have very erroneous notions of what words mean, so it is well in the beginning to advise the writer of a specification to make frequent reference to a comprehensive dictionary, that he may know whether his words are capable of two or more interpretations; if they are, his specification is a failure. A comprehensive dictionary gives illustrations of the proper use of words from the works of eminent writers, and if these are carefully studied, much will be gained in clearness of expression.

Words, names, and terms should be employed only when they are exact. Terms having different meanings in different localities should be studiously avoided, in order that there will be no ambiguity in meaning.

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The aim of the writer should be brevity, by which is meant the employment of only sufficient words to clearly indicate the intention of the writer. This, however, does not mean that any word should be omitted which is necessary to a complete understanding of the requirements.

2. The **specification** is the medium the architect or structural engineer uses to make known to the bidder, and subsequently to the contractor, certain ends, purposes, etc. that cannot be shown or described on the drawings, or that can be more conveniently set forth in a written document, than by notes or lines on a drawing. It is the bill of particulars and should contain all the conditions of contract, such as the limit of time for completing the work, the terms of payment, the responsibility of the contractor for damages to persons or property, etc., commonly called the "general conditions," and any special condition that may affect the cost of the work. It is the memorandum that the bidder uses to make up his estimate, and, therefore, should contain any and every particular that may affect the cost of the operation. The bidder, in making a tender to do the work, bases his proposition on the specification and drawings, and in order that the offer may be a fair one, all the particulars should be shown.

A specification that is vague, indefinite, and insufficient is a reflection on the intelligence of the writer, a source of annoyance and trouble to the bidder and contractor, and a very frequent cause of loss to the owner or the contractor, or to both. Some designers draw up a specification in such a vague way that the contractor is misled in bidding, and a subsequent performance, wholly beyond that understood, is exacted. It requires no extended argument to prove this method peculiarly dishonest.

3. The first aim in writing a specification should be *fairness;* outside of the moral quality involved in it, fairness is a quality that is conducive to good work. If by a defective and careless or loose wording of a specification a bidder is led to underestimate the cost of the work, the injustice of

being required to do more than he figured on causes him to feel justified in slighting parts of the work, if he can. If, on the other hand, the wording of the specification leads the bidder to overestimate the cost, the carelessness of the architect or engineer causes loss to those who have put reliance in his judgment. If the specification is so worded that it is not clear what requirements are to be fulfilled, the contractor, through the ignorance of the architect or structural engineer, may be called on to do things that are useless or unnecessary, for which the owner must pay, thus causing a great waste of time and money.

The drawings and the specifications should be so executed that everything that is to be done or called for is fully set forth in as simple a manner as possible. Some architects and structural engineers, however, claim that if the drawings are complete as to the details, the bids run high, and while this may be true with ignorant bidders, a determination on the part of the designers not to practice evasion in respect to the details would soon establish a standard and competition would soon correct high bidding. With bidders of intelligence and experience, however, this method of evading the details has the effect of raising the level of all bids, by a sum sufficient to cover the uncertainties, or, as the contractors express it, they "add on for what they do not see."

If the drawings and specifications are complete, plain, simple, and straightforward, the designer will find that he has saved himself much trouble that might subsequently arise; if the drawings and specifications are incomplete and uncertain, these uncertainties may lead to differences with the contractor and the owner, and if injustice be done to either, resentment may follow, and the reputation of the designer for intelligence and fairness may suffer.

4. The architect or structural engineer's position must be an impartial one, and he should know neither fear nor favor; he has no right to give to contractors that for which the owner will pay, neither has he the right to give to the owner anything that belongs to the contractor. In the

determination of matters affecting the relative rights of the parties to the contract, the specification plays an important part.

If the designer fails in any degree to fulfil the trust reposed in him, he causes injustice to one party and perhaps to both. He also injures his own character, and lessens the respect in which his profession is held, and thereby brings injury and distrust on men in the same calling whose aims and practice may be on the highest plane of fair dealing.

5. The writer of specifications must be familiar with the qualities of the various materials he proposes using, and must know how best to employ their virtues and avoid their shortcomings. For instance, a stone may have good color and texture and yet not stand the weather; and again, oak has a beautiful grain and is strong, yet will warp on exposure to alternate dampness and drying.

He should be careful to avoid impracticable requirements, but should not hesitate to improve on precedent. Before he specifies anything out of the ordinary, he should satisfy himself of its necessity and of its practicability. Improvements can frequently be made in the prosecution of the work by conferences between the several parties concerned, and the execution of the operation thus greatly facilitated, the designer as an expert constructor being the chief authority in such a consultation. It is advisable for the writer of a specification to consider each part of his specification, and see if he has described the best method of executing the work. If a better way occurs to him and he is not entirely sure of its feasibility, let him discuss it with those interested in the work, in whose judgment he has confidence, and out of the discussion a better method may be evolved than was formerly planned.

6. It is well, in writing a specification, to enumerate the various things required to be done in a logical and ordinary manner, even though many of them may be clearly shown on the drawings. As the drawings and specifications are the two parts of the description of the work about to be

undertaken, each part should be as complete in itself as possible. Any note that pertains particularly to the drawing and will help in carrying out the work should be shown on the drawings, as well as noted in the specification. The specification is the memorandum, and the drawings the graphic diagram of the work; therefore, while the specification is the notice to the bidder of certain requirements that affect the manner and cost of doing the work, it frequently happens that the drawings are given to various workmen, without the specification so that errors are likely to occur if certain notes affecting the drawings are not stated on them.

The various parts of the work done by different subcontractors should be clearly defined, so that they will bid only on the work they are to do and not on a part that may not be required of them.

7. Another very important object in writing a specification should be the prevention of errors arising or occurring in the work, for while it may be easy sometimes to place the responsibility for an error, and while workmen are sometimes careless and short-sighted and very prone to err, errors do no one any good; and while it may be possible to rectify errors at the expense of whoever is responsible for them, they frequently cause great waste of time and material, which always means a loss to some one. Therefore, if it is possible for the writer to anticipate errors and call attention to them before they happen, he will not only be simply doing his duty, but he will improve his capacity for good work by discovering flaws in his own work, or discovering ways to improve the methods he formerly thought were unobjectionable.

Placing on the contractor the responsibility for errors that may arise from a workman's failure to understand the requirements, may find justification in the designer's mind by the thought that he has done his full duty when he indicated what was required, and he cannot hold himself accountable for the errors of others; yet, the aim of the designer is to

secure the best results from the materials at hand. It is an old axiom that the best workman is he who can make the best job with the poorest tools, so, the most successful designer is he who can make the best use of the facilities afforded him by the conditions of the market. If one of these conditions be inefficient or careless workmen, and if by thinking for them he can improve the quality of their work and thereby secure better results, he is doing a great service in preventing waste and loss, and in keeping down the cost of subsequent work, by preventing the cost of possible errors being added to the cost of execution.

An appreciation of the manner in which work is carried on in a commercial way will show the necessity of this forethought in preparing the plans and specifications. In order to carry on a large operation it is necessary to subdivide the work, and a certain amount of dependence must be placed on each workman; if, with the same force of workmen engaged on work of similar character, one set of drawings can be executed without error, and with another set numerous errors occur, it needs no argument to show which is the better set of plans and specifications.

The designer is, or should be, the one most familiar with the plans and therefore the one best fitted to indicate in a clear and graphic way the requirements, and having studied out the problem, he is wasting the results of his own work if he fails to allow others to profit by his labor, so far as it may be possible for him to embody the results of it in the plans and specifications.

8. The specifications usually provide that in case of doubt or difference of opinion arising as to the interpretation of the drawings and specifications, the architect or structural engineer's decision shall be final and binding on both parties.

The relation of the designer to his drawings and specifications and to his client and the contractor is a delicate and difficult one; he may unwittingly err and do an injustice to one or both of the parties concerned. It is evident, therefore, that he must have a judicial temperament.

It is a peculiar position, and contrary to all principles of judicial procedure, that the drawer of an instrument should also be its interpreter, and that his interpretation should be without appeal; for while it is true that the designer may know best what he meant to express by his own words, yet the writing loses this personal quality when it becomes the medium of contract. Words have a specific meaning to the scholar and a general value to the layman, and a wide difference may exist between the two values. The aim, therefore, in writing a specification, should be to avoid the use of words or the manner of using words from which two interpretations can fairly be drawn. In order to illustrate this, consider the word "provide" which, when applied to *labor*, is sufficiently explicit, but when used in connection with *materials* is indefinite, for it does not signify that the material shall be set or put in place, but only that it shall be furnished.

FORMS OF SPECIFICATIONS

HEADING AND GENERAL-CONDITION CLAUSE

9. To facilitate the writing of specifications, and to assist the student in this direction, the following typical specifications are given. The specification in each case is distinguished from the general matter by being printed in smaller type. All words in heavy-face type contained in the body of the specification are defined in the Glossary at the end of this Section, and all dimensions enclosed in parentheses are provisional and subject to changes demanded by the different classes of work or conditions to be fulfilled. Should the different classes of work be made the subject of a separate contract, the specification for each contract should be distinct, and preceded by a heading and a general-condition clause. The heading for the first specification, which is for heavy masonry work, is as follows, and the general-condition clause for this specification will adequately answer for the subsequent specification covering fireproofing.

Specification For Heavy Masonry

Specification of the materials and workmanship required, and to be furnished, for heavy masonry to be constructed for______________
____________________________ at ______________________
in the City of__________________County of______________
and State of__________________in accordance with drawings and this specification, and under the supervision of________________
_______________________, Structural Engineer.

____________________________Address
____________________________Date

10. The **general-condition clause,** which should precede the specific description of the work and material required by every specification, is introduced as a precautionary measure, to cover all those details and conditions that are likely to arise in the progress of building erection, and that cannot be foreseen and provided for under the classified headings of the different parts of the specification. It is affixed to the specification, so that each person who bids on the work will thoroughly understand his risks and responsibilities, and provide accordingly in his quoted price, so that when he signs the final contract, he will be unable to claim that he did not understand the requirements of the owner, and thus be compelled to make an addition to his price to provide for the stipulation afterwards inserted. It is usually stipulated in the contract that the general-condition clause, as well as the whole specification, forms a part of the agreement, equally as binding as the contract itself.

The wording of this clause is very important, as, in the event of a lawsuit over the contract, the provisions of the general-condition clause are likely to become the basis of over half the arguments of the case.

Many of the lawsuits that occur in building transactions are caused by misunderstandings brought about by omissions from, or obscure statements in, the general conditions; hence, this clause should cover all possible contingencies, and provide for them in such clear and concise language that there can be no possible room for doubt in regard to their meaning. It is also advisable to leave a few blank lines at the end of

the general-condition clause, so that special stipulations can be inserted should occasion demand it.

Care should be exercised to see that the general conditions are included in the contract, that is, they may actually be a part of the formal contract; or, as is usually the case, they may form a part of the specifications, especially when they are printed on separate sheets and attached thereto. It should be expressly provided that they are to form a part of the specification, and as such are included in the contract. This precaution, though possibly unnecessary, removes all room for subsequent argument.

11. All practicing architects and structural engineers who have to deal with their clients' bills, appreciate the necessity of providing in the contract that all transportation shall be prepaid. The material dealers and others furnishing goods that are to be sent from a distance, are often inclined to send them by express, and charge the expressage to the owner; and even where there is no intention of compelling the owner to pay transportation, goods are often sent by freight at the owner's expense, and he is supposed to make a back charge on the shipper's account. This practice causes unnecessary bookkeeping for the architect, structural engineer, or owner, and should not be tolerated when it is possible to prevent it. It is also advisable to insert a provision in the general conditions to the effect that, before any part of the work can be sublet, the written consent of the owner shall be required. The architect or structural engineer sometimes reserves this right to himself on the ground that, being more familiar with the work to be executed and the standing of the various contractors, he is more capable to judge of the fitness of the party than the owner. This is not, however, always a wise stand for the architect or structural engineer to take, because it tends to destroy his position as a disinterested adviser, and gives unscrupulous contractors an opportunity to make insinuations of dishonest partiality toward certain mechanics.

The architect or structural engineer would do well to hold his position distinct and separate from the owner, who is one of the

contracting parties. In doing so, he will establish himself as an expert judge, rather than the owner's agent, which is always advantageous, and adds dignity and weight to his decisions.

12. Supervision of Workmen.—The architect, structural engineer, or his assistant should observe the workmanship of the individual men employed by the contractor, so as to guard against the evils arising from the employment of careless or incompetent workmen. Hence, it is advisable to insert in the general conditions a stipulation that will give to the architect, structural engineer, or his representative the power to dismiss such workmen.

13. Contractor's Guarantee.—The contractor should guarantee the work for some stated period after it is completed and accepted, in order that he may, without additional cost to the owner, make good any defects that may have been overlooked. This will avoid a possible lawsuit, where the jury would have to decide the scope of the implied warranty, which exists in all contracts, and how long it held good. Where it is desired to provide this stipulation in the general conditions, it may be stated somewhat as follows: "The contractor is also, at his own cost, to amend and make good any defects, settlements, shrinkage, or other faults in his work, arising from defective or improper materials or workmanship, which may appear within 1 year after the completion of the work."

14. Payment of Building-Permit Fees.—There is always some expense attached to the carrying out of the rules governing works of construction in towns and cities. Outside of the mere expense of securing city permits, etc., there are often the numerous and somewhat changeable wishes of the building inspector to be complied with. A stipulation covering the contingency of such expenses may be inserted in the final contract, though it would be better to place it among the general conditions. The contractor, in bidding on the work, can then take into account the cost that he may incur in complying with such a stipulation.

15. Care of the Work During Erection.—The care of the work during the erection should be properly provided

for in the general conditions, fixing the responsibility on the contractor, and stipulating that "he shall provide all necessary guards, rails, and night lights, and be responsible for all property injured upon, or stolen from, the premises."

16. Time Limit on Work.—Where it is desirous to have the work completed by a certain date, it is well to so stipulate in the general conditions, as well as in the contract, for then the contractor will not have the excuse that when he bid on the work he did not know there was to be a time limit. It is much better for the architect or structural engineer, in fixing the time limit, to divide the construction of the building into stages, and to fix successive dates at which each stage shall be completed, instead of stipulating only the date for the completion of the entire job, for in this way he will be better able during the progress of the work to determine whether the contractor is likely to finish on time or not. That is, if the contractor shall fail to have the first or second stage of the work done at the stipulated time, the architect or structural engineer can notify the owner and have the contract rescinded, and it will not be a difficult matter to prove that the contractor has been negligent. Where, however, only the final date of completion is stipulated, and the architect or structural engineer, judging from the appearance of the work, has reason to believe that the contractor will not finish in the stipulated time, he may remonstrate with him, but probably the only result will be that he will receive the assurance of the confident contractor that the work will be completed at the specified time; and should the architect, structural engineer, or owner rescind the contract at an early stage of the work, he might be called into court for damages, and be required to prove the incapacity of the contractor to finish on time—a thing that might be difficult to do.

17. The general-condition clause for the first typical specification, which relates to heavy masonry construction, is as follows; it will also answer for the subsequent specification covering fireproofing, but a revised general-condition clause is affixed to the specification for steelwork.

SPECIFICATION FOR HEAVY MASONRY

GENERAL CONDITIONS

18. Prices.—The prices given in the accepted proposal shall include the supply and erection, in a good, sound, substantial, and workmanlike manner, of all the items required for the completion of the stone or brick masonry in the proposed work, and shall include all the items shown on the drawings and described in these specifications; also all forms, centering, false works, tramways, machinery, scaffolding, labor, workmanship, tools, and materials necessary and best adapted to the efficient, prompt, and safe execution of both the temporary and the permanent works.

The contractor shall furnish, set, erect, and build all the stone and brick masonry indicated or referred to in the accompanying drawings and these specifications, and when the erection is completed, shall remove all materials and devices used in performing the work, leaving it in a perfect condition. He shall likewise follow the exact details, methods, and instructions called for by these drawings and specifications, and such supplementary details and instructions as may hereafter be made.

19. Drawings.—Drawings illustrating the work consist of the plans, elevations, and details, and give the scheme, design, and general details to be followed. They will be supplemented by such additional drawings and details as may be necessary to carry out the proposed work as set forth in these specifications.

Should any figure be omitted from the general plan or from the details, or should any error appear in either, the contractor shall apprise the architect or structural engineer of such omission or error, and shall under no circumstances proceed with the work in uncertainty or without further detail drawings.

Any portion of the work insufficiently expressed, or not shown for the sake of brevity, on the general or detail drawings is to be completely executed in the full spirit and meaning; and no deviation from the approved drawings and these specifications will be allowed, unless written permission shall have been previously granted by the architect or structural engineer. On the drawings, figures shall always have preference over the scale, in dimensions.

The right is reserved to make changes in the details, in accordance, however, with the general scheme as set forth in the accompanying plans and specifications. If such changes are made, after any special material has been provided, an equitable allowance will be made if such change requires additional work or material. All measurements

must be verified by the contractor from information to be obtained by him directly, or to be had upon application to the architect or structural engineer before proceeding with the work.

In case of any doubt or difference of opinion as to the true intent of the drawings and specifications, the architect or structural engineer must be notified at once, and his decision shall be final and binding on both parties to the contract.

20. Time Limit.—The time required by the bidders for the completion of the work from the day of the acceptance of the proposal must be specified in the bids and the date of completion will be inserted in the contract. In case this time is exceeded, the owner is hereby authorized to deduct and retain out of the payments that may be due, or become due, the sum of——————dollars, as liquidated damages and not as penalty, for each and every day occupied beyond the time so stipulated.

The contractor will be required to give security, in form to be approved, in the sum of one-half the amount of his contract, for the faithful execution of the work.

21. Additional Work.—Whenever, in the opinion of the architect or structural engineer, work that is neither contemplated in the plans nor implied in the specifications referring to said plans, shall become necessary, the contractor hereby agrees to furnish such materials and perform such labor as extra work, under such terms as may be agreed upon in writing, without in any way violating the terms of this agreement; and in case terms cannot be agreed upon, before the work is undertaken, the contractor shall proceed with such extra work upon the written order of the architect or structural engineer and the proper value of such extra work shall be determined by three arbitrators, one to be chosen by the owner, one by the contractor, and one to be chosen by the two arbitrators so named, the award of such arbitrators to be binding on both parties to this agreement.

Should the performance of this extra work require additional time, this time shall be agreed upon in writing, or submitted to the arbitrators previously referred to.

No claims for extra work of any kind will be allowed unless such extra work shall have been ordered in writing by the architect or structural engineer, and the price agreed upon, or unless the work has been so ordered in writing, and the value left for determination by arbitrators under the conditions previously set forth.

22. Payments on Account.—Payments on account of the work executed will be made on estimates furnished by the architect or structural engineer on or about the first day of each month. These estimates will be based on the amount of work done during the preceding

month, and a certificate will be issued to the contractor, setting forth the value of this work, with an order on the owner for 80 per cent. of its value, which order shall be payable on or about the tenth day of the month. The balance shall be retained each month until 60 days after the completion of the work, and the acceptance of the structure, as a whole, by the architect or structural engineer, subject to all legal or equitable deductions.

23. Responsibility of Contractor.—Responsibility for all risks and casualties shall be assumed by the contractor and he will be held responsible for all accidents, negligence, or carelessness pertaining to his work or to the material used, and he must make good, without delay, any damage arising therefrom. He must use every precaution to protect the workmen and others about the operation and the public on the streets, and he shall indemnify and save harmless the owner from all suits or actions of every name or description brought against him for, or on account of, any damages or injury received or sustained by any party or parties, by or from the contractor, his agents, or servants in the performance of the work.

The contractor will be held responsible for any and all violations or infringements of patent rights, and save harmless the owner from any loss or damage, by suit or otherwise, for any device, process, or materials used in the construction of the work called for in the drawings and these specifications.

The contractor shall be held responsible for any damage that may arise from delays in the execution of his contract, or any unnecessary obstruction to the work of other contractors. He shall also be responsible for observing all City Ordinances and Acts of Assembly. He must pay all fines, expenses, etc. that may result from obstructing the streets, and he shall pay all proper and legal charges to public officers for permits.

The contractor shall also maintain at all times a sufficient amount of insurance to protect the owner from loss, in a manner satisfactory to the owner.

The contractor shall have charge of, and be responsible for this work until it is completed and accepted by final payment; he shall give his personal supervision for the faithful prosecution of the work, shall not sublet or assign the same, but keep it under his own control; he shall also have a competent representative or foreman on the work, and shall receive orders and directions from the architects, structural engineers, or their inspectors, and shall have full authority to execute such orders or directions without delay, and to immediately supply materials, tools, and labor, as may be required.

The contractor shall employ only competent and efficient laborers and first-class mechanics or artisans for every kind of work, and whenever in the opinion of the architect or structural engineer any man is

unfit to perform his task, or does his work contrary to directions, or conducts himself improperly, the contractor must discharge him immediately and not employ him again on the work.

The contractor shall cooperate at all times with other contractors and subcontractors on the work, so that when the work is completed it shall be in accordance with the design and a complete and finished piece of work.

24. Inferior materials or workmanship not in accordance with the finally approved drawings and these specifications, brought to or incorporated in the work, shall be immediately and entirely removed by the contractor from the vicinity, and built anew; and if the directions of the architect or structural engineer are not complied with after written notice, he shall be at liberty to remove the same at the expense of the contractor, and the cost thereof shall be deducted from any money which may be due him.

Materials and workmanship may be reinspected at any time.

25. Supervision of the Work.—The supervision of the work shall at all times be entrusted to the architect or structural engineer and his authorized assistants; they shall have free access to the work and every facility shall be afforded them for inspection.

FOUNDATIONS

26. Removals.—All refuse and surplus material shall be removed from time to time, as directed, and no part shall be considered removed that has been thrown or shall accidentally fall into a creek or river. Rock surfaces for foundations shall be made approximately horizontal. All parts of **coffer dams** and other temporary works shall be removed, unless otherwise directed by the architect or structural engineer.

27. Temporary Work Left in Place.—Should any part of the coffer dams or other temporary works be left in place by direction of the architect or structural engineer, no charge for the same shall be made by the contractor.

28. Handling Material.—The materials excavated shall be handled in the most approved manner for transporting the same so as to avoid blocking highways with surplus earth. The contractor, when so ordered by the architect or structural engineer, shall make any change in the method of excavation that shall be deemed necessary for the protection of property or maintaining travel.

29. Blasting.—No blasting will be allowed unless the contractor first obtains a permit from the Department of Public Safety,

through the Director of the Department of Public Works, and then only on condition that all blasting powder, dynamite, etc. shall be kept in a secure and approved manner and shall be at all times under the special care of a watchman. Each blast shall be covered with heavy timber or mats before firing, when so directed. It is understood that all blasting is to be done in such a manner as will comply with all laws and ordinances relative to the storage and use of explosives, and care must be exercised to see that neither life nor property is endangered. The blasts are to be fired at such times as may be ordered, and whenever directed the number and size of the charges shall be reduced; no claim for loss or delay will be allowed on this account. The explosive to be used shall be subject to the approval of the architect or structural engineer.

30. Prosecution of Work.—When quicksand, running sand or other treacherous material is found, the work shall be executed with the utmost despatch and vigor, and proceeded with continuously day and night, if so directed.

31. Foundation Footings.—Should the materials upon the site not be considered sufficiently firm and solid, they shall be removed until satisfactory natural foundations are reached; or artificial foundations may be constructed of timber pilings or platforms, or both, as directed, and extending in width and length at least 6 inches beyond the footings of that part of the structure to be placed on them. Concrete shall be used if so directed. In any case, if work of this character be required, the contractor will be paid for the same as herein provided. The contractor shall suspend work to allow examination and explorations to be made.

32. Piles.—The piles shall generally be arranged as shown on the drawings; in all cases they shall be driven over the entire required areas, in straight rows, not more than 3 feet apart between centers. They shall be of sound and thrifty white oak, long-leaved yellow pine, or spruce pine, as directed, of straight growth. They shall be 2 feet longer than the necessary length, not less than 8 inches at the small end, and not less than 12 inches at the butt end, when sawed off. They shall be barked, trimmed close, pointed, and hooped to prevent splitting in driving, shod if required, and driven to refusal, or as directed, with a hammer weighing at least 2,000 pounds. The tops of all piles after driving shall be sawed off truly to the level of grade, as required.

Raking or batter piles shall be driven and bolted where and as required.

33. Platforms.—The platforms shall be placed directly upon the earth or upon the piles. The timber shall be squared yellow pine

or hemlock, as directed, generally 12 inches by 12 inches in cross-section, the first course laid parallel with the longest side of the foundation, not more than 3 feet apart between centers. The space between these timbers shall be filled with stones of suitable size, well rammed, or with concrete, and accurately leveled off. Upon these shall be laid, in a transverse direction, a second course of the same kind and section of timber, laid close, and of sufficient length to cover the first course. These timbers shall be securely fastened to the piles and to each other by 1-inch, rough, wrought-iron or steel drift bolts, 24 inches and 20 inches in length, respectively. These sizes and their arrangement may be changed when so directed, and under all conditions shall be so designed as not to exceed the unit stress demanded by good practice.

STONE MASONRY

34. Stone.—The stone masonry shall be built of the designated stone to the dimensions and cross-section, and with such arrangement of courses and bond as are directed or shown on the drawings. The stone shall be hard and durable, as large as practicable for the intended work, of approved quality and shape, and in no case having less bed than rise. The stones shall be laid on their natural beds, and shall be well bonded and solidly bedded, all spaces being filled with stone chips and mortar, or thoroughly grouted as required. The first stones laid in the foundations shall be large, selected, flat stones.

All masonry shall be laid up by being thoroughly bedded in cement mortar, and the face and back of the wall shall be carried up together over the entire wall, in approximately the same total height.

35. Dressing.—In rock-faced ashlar work, the faces of the stones shall have uniform projections not exceeding (2) inches, and in rough-pointed ashlar work not over (1) inch beyond the neat lines, and in both cases they shall be pitched to a straight line at all joints. In fine-pointed ashlar work, the faces of the stones shall have no projections beyond the neat lines, and shall have a **draft** not less than (1) inch wide at each joint.

In rubble masonry, the faces of the stones shall have no projections more than (3) inches beyond the neat lines or faces of joints when pointed.

All projecting angles and **arrises** shall have a (1½)-inch draft on each face.

Dressed stone is to be set with a **lewis**, if required, and where it is fully bedded in mortar it shall be settled on the bed with a wooden mall when directed.

36. Capstones and Truss Seats.—Truss seats and capstones for column piers shall be of approved granite, bluestone, or

syenite, of the thickness shown. The top surface under the bearings of trusses, main girders, columns, or other principal metal bearings shall be **bush-hammered,** and the remainder of the upper surfaces shall be **peen-hammered,** except the caps of column piers, which shall all be bush-hammered on top. The showing edges of truss seats and capstones shall be **rough-pointed** when the masonry is rock-faced, **fine-pointed** when the masonry is rough-pointed, and peen-hammered when the masonry is fine-pointed. All stones in seats and capstones shall be **tooled** over the entire beds.

37. Coping Stones.—Coping on ashlar-faced masonry shall generally be of the same kind of stone as that used in the ashlar, or of granite, syenite, or bluestone, if shown. Coping on rubble-masonry retaining walls shall be of the same kind of stone required for ashlar masonry, or of granite, syenite, or bluestone, if so shown. It shall be (20) inches wide, (8) inches thick, as shown on the drawings, with a drip line, a wash (½) inch thick, bush-hammered on top with five cuts per inch and peen-hammered on showing edges. The coping stones shall be dressed to a straight line on the back, and to a sufficient depth to receive abutting pavement. They shall be in as long lengths as practicable, but not less than 4 feet, and shall have the ends square-jointed, notched, cut to bevels, and doweled or clamped, as required.

38. Finish of Wall Not Coped.—The top of the masonry, when not covered with coping, shall be covered with large stones selected for the purpose, to give an approximately flat surface. Parapet walls of rubble masonry shall generally be (18) inches thick, and when not covered by a cut coping, shall be finished with a rustic combing, formed of stones with the corners broken off and set on edge in mortar and pointed.

39. Pointing.—The joints on faces of masonry shall be cleaned out to a depth of 1 inch, wetted and pointed with Portland cement mortar securely pressed into the joints; the whole work is to have a neat and clean finish. Cut-stone work is to be **hollow-pointed,** if so directed; all other masonry, **cut-pointed.**

40. Drilling, Etc.—All drilling for bolt holes, and all cutting and dressing necessary shall be done before the stones are set, and by skilled workmen, at the expense of the contractor; under no circumstances will the hammering or dressing of stones upon the walls be allowed. The stones must be placed in position so as not to disturb those previously laid.

41. Wetting of Walls.—All walls shall be kept wet as the work progresses, when so directed.

42. Drainage of Masonry.—Before filling in behind abutments, wing and retaining walls, a drain arranged to discharge into 3-inch cast-iron pipes built through the masonry where and as directed, is to be constructed for the purpose of draining the back filling.

43. Dashing Backs of Walls.—The joints on the back of all abutments, wing and retaining walls shall be carefully and thoroughly dashed with cement mortar, so as to make the walls water-tight.

44. Dry Walls, Riprap.—Stone may be laid dry without mortar, by hand, when so directed; dry walls and **riprap** shall generally be of stones from $\frac{3}{4}$ cubic foot to 2 cubic feet in volume.

45. Rubble.—The stone used in rubble masonry shall consist of first-class stone, or other approved stone of good shape and flat beds. No stone shall have less bed than rise and it must be laid on its broadest bed in cement mortar.

46. Headers and Face Stones.—The headers shall generally form at least one-fifth of the faces and backs of walls, with a similar number distributed through the mass where they do not interlock, and the face stones shall be well **scabbled,** so that they may be set close; **chinking** with small stones is to be avoided.

47. Sizes of Stones, Etc.—In walls 5 feet thick or less, the stones used shall average from 6 to 8 cubic feet in volume, and the lengths of the headers shall be equal to two-thirds of the thickness of the wall; in walls over 5 feet in thickness, the stones shall average 12 cubic feet in volume, and the headers shall not be less than 4 feet long. Generally no stones having a less volume than 4 cubic feet shall be used except for filling the interstices between the larger stones.

48. Limit of Height and Bond.—In no case shall stones having a greater height or build than 30 inches be used. Stones approximating this size must bond the joints above and below at least 18 inches; in all other cases the smaller stones used must bond the joints above and below at least 10 inches.

49. Range Ashlar.—In range ashlar, the face stones shall be uniform in color, free from stains, and if facing a street shall also be free from drill or dog marks. All mortar or grouting that has run out over the faces must be washed off clean before it has set. The faces of the headers in each course must form at least one-fifth of its face, unless otherwise shown.

50. Courses.—The height of the courses shall not exceed (20) inches nor be less than (16) inches. The stone shall be dressed for

½-inch joints, true to the proper lines, out of wind, with parallel beds and vertical joints. The vertical joints shall be dressed at least 6 inches back of the face, and the beds shall be tooled over their entire surfaces.

51. Headers.—The headers must have as much width of face as rise, and they shall generally be not less than (3) feet long. This latter dimension is an approximate average length; headers in the bottom of all walls shall be of a greater length, and they may be shortened as their positions approach the top, in proportion to the dimensioned thickness of the wall.

52. Stretchers.—The stretchers shall have as much bed as rise, but shall not be less than (14) inches wide; they shall not be less than (2) feet long, and shall not break joint on headers.

53. Backing.—The backing shall be of coursed rubble masonry well bonded with the ashlar and in itself.

54. Arches.—1. *Voussoirs.*—Arch stones shall be laid up with freshly mixed Portland cement mortar, in regular courses, with their beds in radial planes, with bonds on contiguous courses of not less than (12) inches. They shall be dressed for ½-inch joints, tooled over their entire beds, and shall be dressed (6) inches back of the face at the head-joints, and fine-pointed on the face, or **intrados.**

2. *End Ring Stones.*—End ring stones shall project 3 inches beyond the **spandrel** faces, and they shall be rough-pointed on the ends and fine-pointed on the intrados.

3. *Backing.*—The backing of arches shall be of rubble masonry, laid in cement mortar, having its top and end surfaces brought smooth by plastering with cement mortar to shed water; this plastering shall extend up on the inside faces of the spandrel walls at least 12 inches.

These plastered surfaces, except those of the ends of the backing, shall have, in addition, a 1-inch coating of **asphaltic mastic** equal to Neuchatel asphalt mastic.

Drainage outlets, of heavy leaden pipes 1½ inches in diameter, shall be placed over piers and through arches when required.

The intrados of all stone arches shall be scraped or otherwise cleaned free of mortar, and the joints pointed immediately after the centers are struck. The intrados of brick arches shall be plastered with Portland cement finished by floating.

BRICK MASONRY

55. Bricks, Etc.—The brick masonry shall consist of whole, sound, straight, hard bricks, laid in freshly mixed cement mortar to the forms and sections required. The bricks shall be thoroughly

wetted and laid true to line in parallel courses, properly bonded, with face joints flush, and not exceeding $\frac{1}{4}$ inch, and struck with the point of the trowel or pointed. Every brick must be laid in full, close joints of mortar on its bed, end, and side, at one operation.

The best of the bricks shall be selected by the contractor and used for face work. Broken bricks, not less than halves, may be used, if directed, in places not affecting the strength of the work.

Brick arches must be bonded as shown by the drawings, and they shall not be built in continuous concentric rings. The arches shall in all cases be laid up with cement mortar. Every seventh course throughout shall be a header course.

MORTAR AND GROUTING

56. Proportions.—All mortar to be used in the building of masonry shall be composed of 1 part of Portland cement to 3 parts of sand. The mortar for grouting, pointing, and bedding copings shall be composed of 1 part of Portland cement to 2 parts of sand. All of these mixtures shall be proportioned by measurement, not by estimation, and shall be first thoroughly mixed dry in suitable tight boxes, after which the proper amount of water shall be gradually added.

Only such quantities of mortar or grouting as are needed for immediate use shall be mixed; if allowed to set, it shall not be retempered and used in any masonry construction.

57. Sand.—Sand for grouting shall be tide-washed, sharp, silicious, dry-screened bar or approved flint-bank sand, free from all loam, dirt, or dust.

Sand for mortar shall be composed of grains graded from coarse to fine, thoroughly screened to reject all particles exceeding $\frac{1}{4}$ inch in diameter, and shall be clean and sharp, free from loam, dirt, or dust, and equal in quality to the best bank sand.

58. Water.—The water shall be fresh and free from dirt. Salt water may be used, when directed, when necessary to construct masonry in cold weather.

59. Tensile Strength.—Mortar taken from the mixing box, and molded into briquets 1 square inch in cross-section, shall develop the following ultimate tensile strengths: 7 days (1 day in air, 6 days in water), 1 part of Portland cement to 3 parts sand, 125 pounds; 28 days (1 day in air, 27 days in water), 1 part of Portland cement to 3 parts of sand, 175 pounds.

CEMENTS

60. Kind.—Portland cement shall be used in all the mortar, concrete, and grouting required by these specifications.

61. Inspection.—All cements must be inspected, and those rejected shall be immediately removed by the contractor. The contractor must submit the cement, and afford every facility for inspecting and testing, at least 12 days before desiring to use it. The architect or structural engineer shall be notified at once upon receipt of each shipment at the works.

62. Packages.—No cement will be inspected or allowed to be used unless delivered in suitable packages properly branded.

63. Protection.—The accepted cement must be protected in a suitable building having a wooden floor or platform raised from the ground, and may be reinspected at any time.

64. Failure.—The failure of a shipment of cement on any work to meet the following requirements may prohibit further use of the same brand on that work.

65. Acceptance.—The acceptance or rejection of a cement to be used shall rest with the architect or structural engineer, and shall be based on the following requirements.

66. Fineness.—Portland cement shall have a specific gravity of not less than 3.1. The residue, by weight, on a No. 100 sieve shall not exceed 10 per cent.; on a No. 200 sieve, 25 per cent.

67. Sieves.—Sieves shall be made of brass-wire cloth, having approximately 2,400, 10,200, and 35,700 meshes per square inch, the diameter of the wire being .0090, .0045, and .0020 inch, respectively.

68. Constancy of Volume.—Pats of **neat cement**, ½ inch thick, with thin edges, immersed in water after **hard set**, shall show no signs of checking or disintegration.

Pats of neat cement, 3 or 4 inches in diameter, ½ inch thick at the center and tapering to a thin edge, shall be preserved in moist air for 24 hours; they shall then be exposed for 3 hours in an atmosphere of steam coming from boiling water in a loosely covered vessel, and shall show no signs of checking, cracking, distortion, or disintegration.

69. Sets.—It shall not develop **initial set** in less than 20 minutes, this being determined by means of the **Vicat needle** on pats of neat cement of normal consistency, the temperature being between 60° and 70°.

70. Anhydrous Sulphuric Acid.—It shall not contain more than $1\frac{75}{100}$ per cent. of anhydrous sulphuric acid, SO_3.

SPECIFICATIONS FOR HOLLOW TERRA-COTTA FIREPROOFING

71. Material.—All material shall be made of a proper mixture of fireclay and sawdust before burning. The use of charcoal as a filler will not be permitted. The finished material shall be evenly burnt, sound, and free from imperfections, and shall be obtained from a maker of established reputation. It may be of two kinds—porous and semiporous.

72. Floor Arches.—All floor arches shall be of semiporous material (end construction).

All skewbacks shall be formed to fit the beams and shall have flanges molded on the skewback, projecting $1\frac{1}{2}$ inches below the bottom of the beam. The projecting flanges shall have a hollow space in them to form a dead air space to protect the beam. No skewback on which these flanges are broken shall be set. No loose projecting pieces shall be used for the protection of the steel-beam flanges.

The end-construction arches shall be of an approved pattern, that will enable the tie-rods to pass through them without touching the terra cotta, or requiring it to be cut. The sides, webs, tops, or bottoms of these floor arches shall not be less than $\frac{5}{8}$ inch thick. The arches shall in all cases be set with proper camber, on a stiff wooden center, in the most careful manner, and shall be laid up in Portland cement mortar composed of 1 part of cement and 3 parts of sand, and no arch shall be of less depth than the I beams, unless otherwise shown.

73. The thickness of $1\frac{1}{2}$ inches of terra cotta, or any other material used for fireproofing, is not sufficient to properly protect the lower flange of the I beams in the event of great heat. The specifications here given cover the standard form, but this thickness should be increased to not less than 3 inches. The covered beam projecting below the flat part of the ceiling can be suitably treated in an architectural manner or simply finished as a beam.

74. Concrete Filling.—The tops of arches shall be filled with cinder concrete up to the under side of the finished floor. If the finished-floor surface is to be wood, 2″ × 3″ beveled sleepers shall be set in the concrete. Cinder concrete shall be mixed in the following proportions:

American Portland cement, first quality	1 part
Clean, sharp sand or fine gravel free from clay .	2 parts
Clean, washed, hard-coal, boiler cinders	6 parts

In mixing the concrete, the cinders are first to be spread upon a tight platform in a layer about 10 inches in thickness; upon this the sand is to be placed in an even layer, and upon the sand, cement is to be laid in a similar manner; the pile is then to be turned over twice with shovels, and water is then to be added from a spray, after which the whole mass is to be turned over twice with shovels. After the concrete is placed in position upon the tops of the arches, it is to be thoroughly rammed with iron rammers, and its consistency should be such that this ramming will bring a film of water to the surface of the mass.

75. Fillers.—Semiporous terra-cotta fillers may be used on top of the arches to reduce the amount of concrete and the weight, but in no case shall the concrete filling be less than 2½ inches.

76. Fastening of Sleepers.—Where the concrete is not of sufficient depth to hold the sleepers, they shall be fastened directly to the floor tile or secured to the top flange of the I beams.

77. Beam, Column, and Girder Protection.—All beams and girders that project below the ceiling line in any part of the building, and all columns, unless otherwise shown or specified, shall be protected with 2 inches of porous, hollow, terra-cotta fire-proofing. All girder and column fireproofing shall be completely covered with a wire-mesh fabric, carefully secured between the joints in girder covering and the ends carefully tied together in column covering.

78. Wall Furring.—The inside surface of all exterior walls shall be furred with 1½-inch porous terra-cotta tiles. This furring shall be secured to the outer wall by means of wall ties, which are to be put in as the exterior wall is built. It shall be the duty of the contractor for fireproofing to furnish these ties and to see that they are properly placed to suit his work.

79. Roof Tiles.—Porous terra-cotta roof tiles, of the thickness shown, are to be set between the iron purlins to form the roof covering. The porous terra-cotta tiles shall be made and burned so that the nails that secure the tile or slate to them shall have a proper and sufficient "purchase."

80. Testing.—Tests shall be made of any of the spans at the discretion of the architect or structural engineer. The contractor for the fireproofing shall shore up the beams and test the arches with a load of not less than 500 pounds per square foot, without undue deflection. Tests of the fireproof qualities of the floors or partitions may be made in the building or out of it, at the discretion of the architect or structural engineer.

POROUS TERRA-COTTA TILE PARTITIONS

81. All partitions, unless otherwise indicated, shall be of the best make of porous terra cotta, of the thickness shown (either 2, 3, 4, 6, or 8 inches). The thickness of the webs of the partitions shall in no case be less than ⅝ inch. All blocks shall be set in the best manner in Portland cement mortar, composed of 1 part of cement and 3 parts of sand.

All partitions shall be set directly on the floor tile or on top of the beam or girder, and after being set in place the concrete filling shall be thoroughly tamped around them. Before setting any partitions in place, all dust and dirt of every kind shall be removed, water to be used if necessary to insure this end, so that the cement mortar will adhere directly to the material on which the partition rests and not to the dust that ordinarily covers the tile floors in unfinished buildings.

All partitions shall be tightly and carefully wedged against the ceiling or the upper side of the floor tiles of the floor above, with small iron wedges. This work must be done so carefully that the partition will not fall or give way under any circumstances.

82. Door Openings.—All door openings shall be framed with iron studs, securely fastened to the floor and ceiling. The frames shall have iron lintels over openings to support the partition blocks. Studs shall be of such section that they can be securely covered with the partition blocks or special covering. Wooden studs are not to be used under any circumstances.

SPECIFICATIONS FOR EXPANDED METAL AND CONCRETE CONSTRUCTION

FLOOR AND ROOF CONSTRUCTION

83. Centering for Floors.—Centering must be of a substantial character and sufficiently strong to stand without springing when the concrete is laid and tamped upon it.

84. If a smooth, even surface on the under side of the concrete is desired, the boards forming the centering

should be surfaced on the top, of an even thickness, and laid without large or irregular joints.

85. Expanded Metal.—The expanded metal of the mesh and gauge indicated shall be laid directly upon the centering. Adjacent sheets shall be lapped at least one mesh at all intersections, and securely tied with No. 18 iron wire before the concrete is put in place.

86. The object of tying the sheets together is to keep them in place while the concrete is being put down, and is not to bind them together, as this is done by the concrete.

87. Concrete.—The concrete shall be mixed in the following proportions:

American Portland cement, first quality	1 part
Clean sharp sand, or gravel free from clay	2 parts
Clean, washed, hard-coal boiler cinders	6 parts

The manner of mixing the concrete shall be the same as described in connection with hollow terra-cotta fireproofing.

88. Thickness of Floors.—The thickness of the expanded-metal and concrete floor system shall be consistent with the span and materials used, and shall be such that the floor system will safely sustain the combined live and dead loads and will likewise meet with the approval of the engineer and the building department of the city in which the building is to be erected.

89. The thickness of floors necessary to carry a given load on a given span varies, to a great extent, with the quality of materials used and the thoroughness with which the mixing and tamping is done. In Eastern Pennsylvania, where high-grade materials are easily obtainable, a cinder-concrete slab 3 inches thick, with a mesh of No. 10 gauge embedded in it, is generally considered sufficient for a safe load of 150 pounds per square foot upon a span of 6 feet. For longer spans or heavier loads, an increase in the thickness of concrete is necessary, and it is sometimes advisable to use two sheets of expanded metal on very long spans or for very heavy loads.

90. Finished Floor Surfaces.—The finished floor throughout all rooms shall consist of $\frac{7}{8}$-inch, matched, **rift-sawed,** hard-stock yellow pine, not wider than $2\frac{1}{2}$ inches. This finished floor shall

be laid upon $1\frac{1}{4}'' \times 6''$ matched, rough, spruce flooring, which is thoroughly secured to $2'' \times 3''$ sleepers spiked to the cinder concrete of the floor system. At intervals of from 5 to 6 feet there shall be placed between the sleepers, filling ribs of concrete at least 4 inches wide and of the height of the sleeper, the ribs being so constructed as to efficiently act as a fire-stop. In all halls, corridors, and toilet rooms the floor shall be of gray Knoxville marble tiles 10 inches square and 1 inch thick, finished square and true on the edges and rubbed on the face with corners preserved. These tiles shall be laid close in a 1-inch coating of Portland cement with the finished wooden floors of the rooms.

91. When a cement, tile, marble, mosaic, or asphalt finished floor surface is required, such material can be laid in mortar or otherwise, as may be required, directly upon the cinder concrete; a wood-finished floor can be nailed directly to the concrete, but it is usual when a wood-finished floor is desired to nail $2'' \times 3''$ sleepers to the cinder concrete and to place between these, at intervals of 5 or 6 feet, a rib of concrete to form a fire-stop; the wood floor is afterwards nailed directly to these sleepers. In office buildings, apartment houses, etc., where the amount of combustible material is usually so small that a moderate degree of protection for the steel beams suffices, and where flat ceilings are usually desired, an expanded-metal ceiling suspended below the beams is frequently used. In warehouses, and other buildings where a flat ceiling is not necessary, but in which a hot fire may rage for several hours, the steel beams are usually incased in cinder concrete. This should be done at the same time that the floors are laid, so that each bay is a monolith. The lower flanges of the beams should be protected by cinder concrete or cement mortar $1\frac{1}{2}$ inches thick, reenforced by a strip of expanded metal wrapped around the lower flanges of the beam before the plastic material is applied.

92. Roof Surface.—The roof surface shall be graded to drain the water in the proper direction, as designated on the drawings, by cinder-concrete filling. On the top of the filling there shall be placed a 2-inch layer of asphalt in which shall be laid roofing tile of red clay. The tiles shall be free from imperfections as to shape and strength and shall be grouted between the joints with asphaltum cement.

93. In roof construction, tile, slate, or slag covering can be laid directly on the cinder concrete, which should first be graded sufficiently to turn the water in the proper direction.

94. Column Covering.—All free columns are to be incased in expanded metal; suitable light iron furring shall be provided to offset the lathing at least 2 inches from the column, which shall be finished round or square as shown on the plans. The space between the columns and the expanded metal shall be filled solid with concrete and surfaced up ready for plastering.

95. Girder Covering.—All girders projecting through the ceiling shall be incased in expanded-metal lathing. The lathing is to be rigidly supported by suitable light iron furring, and built out so as to offset the expanded metal at least 2 inches from the girder. The space between the girder and the expanded metal is to be filled solid with cinder concrete and the surface left ready for plastering.

SPECIFICATIONS FOR EXPANDED-METAL PARTITIONS

96. Solid Partitions 4 Inches Thick.—The studs shall consist of $2'' \times 1'' \times \frac{1}{8}''$ channels, spaced 18 inches center to center, and securely fastened at the top and bottom. Double lines of studding shall be braced together at intervals of about 4 feet. (Rough wooden frames for all openings will be furnished and set in position under the carpenter's contract.) $2'' \times 2'' \times \frac{1}{8}''$ angle irons, or $2'' \times 2'' \times \frac{1}{8}''$ channels, shall be secured to the vertical sides of all door frames and extend from floor to ceiling. After all the light ironwork and wooden frames are in position, expanded-metal lath, painted on both sides, shall be laced on both sides of the studs with No. 18 galvanized wire. The space between the expanded-metal surfaces shall be filled solid with cinder concrete of the composition specified above, embedding the channels and forming the concrete slab about $2\frac{1}{2}$ inches in thickness. All surfaces are to be left ready for plaster.

97. Hollow Partitions 4 Inches Thick.—Same specification as above, but with cinder-concrete filling omitted.

98. Solid Plaster Partitions 2 Inches Thick.—The studs shall consist of $\frac{7}{8}$-inch channels, or $1'' \times \frac{3}{16}''$ flat iron, spaced 16 inches center to center, securely fastened at the top and bottom. All openings shall be framed with $1'' \times 1'' \times \frac{3}{16}''$ angle iron or $\frac{7}{8}$-inch channels; the vertical members at door openings shall extend from floor to ceiling. All angle-iron framing shall be suitably punched to permit the fastening of the woodwork. When all the light ironwork is in position, painted expanded metal is to be laced to one side with

No. 18 galvanized wire. All necessary wood furrings for base board, chair rail, and picture molding are to be set in position under the carpenter's contract before plaster is applied.

All channel studs are to be bent at a right angle at the lower end to form a foot about $2\frac{1}{2}$ inches long, which must be fastened to the floor, if it is of wood, by means of heavy wire staples or screws. When a stud is to be secured to a steel beam, a strong clip must be used, and when a stud comes over or under cement or brickwork, a hole must be drilled in the brickwork, etc., and the stud wedged tightly in it. The studs must be straight and set plumb in the line of the partitions, and all partitions must be braced before plastering, so that the plasterer will have no difficulty in producing flat, even surfaces.

99. If a strictly fireproof partition is required, no wood should be used for nailing blocks or other purposes. The use of wood for structural purposes in fireproof partitions is not to be commended under any circumstances.

100. Where plaster alone is used as a filling in wire-mesh partitions, the plaster should be lime mortar, strongly gauged with Portland cement. Sulphate-of-lime plasters, which comprise most of the patent plasters, and plaster of Paris exert a corrosive action on metal work.

SPECIFICATION FOR THE ROEBLING SYSTEM

SYSTEM A–TYPE 1

101. Floors.—The fireproofing between the steel floor and roof beams shall be the Roebling System A, Type 1, or arch construction, consisting of a steel-ribbed, wire-cloth centering and a cinder-concrete arch.

The wire centering shall consist of No. 22, four-warp, two-filling wire cloth stiffened with $\frac{3}{8}$- to $\frac{1}{2}$-inch steel rods woven into the cloth at intervals of about 9 inches. This centering shall be sprung in between the I beams in the form of an arch, with the ends of the rods abutting into the seat formed by the web and lower flange of the I beams; the sheets to be well lapped and securely laced.

On the wire centering so constructed, cinder concrete, mixed in the proportion of 1 part of high-grade Portland cement to $2\frac{1}{2}$ parts of sharp sand and 6 parts of clean cinder, shall be laid, providing a thickness of not less than 3 inches at the crown of the arch; the concrete, generally, is to be filled flush with the tops of the floor beams, leaving the floors ready for nailing sleepers.

102. It is advisable in all cases in specifying these several floor systems to insert the following clause:

The flooring shall be subject to test at any points that may be designated by the architect or structural engineer. It shall in all cases develop a strength, in 30 days, equal to the capacity of the supporting steelwork.

103. Ceilings.—The ceilings over the several stories shall be flat supporting rods or bars, spaced 16 inches apart, and shall be securely fastened, transversely, to the under side of the I beams by suitable iron clamps. In all spans over $3\frac{1}{2}$ feet, a $\frac{5}{16}$-inch steel rod shall be laid over and laced to the supporting rods or bars in the middle of the span. To this rod shall be attached supporting wires dropped from the crown of the floor arches, at intervals not exceeding 32 inches. Painted wire lathing stiffened with $3\frac{1}{4}$-inch solid steel ribs woven in every $7\frac{1}{2}$ inches, shall be applied to the supporting rods or bars with the ribs crossing them at right angles; the ribs in the lathing to be securely laced to the supporting rods or bars at every intersection with No. 18 galvanized wire. All ceilings shall be finished ready for plaster.

104. Columns.—All free columns are to be incased by painted wire lathing, stiffened with a $\frac{1}{4}$-inch solid steel rib woven in every $7\frac{1}{2}$ inches. Suitable light iron furring shall be provided so as to offset the lathing at least 2 inches from the column, which shall be finished round or square, as shown on the plans. The space between the columns and the wire lathing shall be filled solid with concrete, and the surface shall be left ready for plaster.

105. Girders.—All girders projecting through the ceilings shall be incased by painted wire lathing, stiffened with a $\frac{1}{4}$-inch solid steel rib woven in every $7\frac{1}{2}$ inches. The lathing shall be rigidly supported by suitable light iron furring, built out so as to offset the wire at least 2 inches from the girder. The space between the girder and the wire lathing shall be filled solid with concrete, and the surface shall be left ready for plaster.

SYSTEM A—TYPE 2

106. Floors for Heavy Construction.—The specification for floors is the same as for System A, Type 1.

107. Ceilings.—The ceilings over the several stories shall be arched or segmental. The beams, in all cases, shall be incased by wire lathing bent to form and fastened to the wire centering on both

sides of the beams, so as to provide a space of at least 2 inches between the beams and the wire lathing. This space shall be filled solid with concrete. The under surface of the arches shall be left ready for plaster.

108. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

SYSTEM A—TYPE 3

109. Floors.—The specification for floors is the same as for System A, Type 1.

110. Ceilings.—The floor beams, in all cases, shall be incased with wire lathing, bent to form and fastened to the wire centering on both sides of the beams so as to provide a space of at least 2 inches between the beams and the wire lathing. This space is to be filled solid with concrete. There shall be provided suitable clamps or hangers for suspending a wire ceiling to finish (6) inches below the beams when plastered. These hangers shall support flat iron bars set on edge and spaced 16 inches apart. In all spans over $3\frac{1}{2}$ feet, a $\frac{5}{16}$-inch steel rod shall be laid over and laced to the flat iron bars in the middle of the span. To this rod shall be attached supporting wires dropped from the crown of the arch, at intervals not exceeding 32 inches. Painted wire lathing, stiffened with a $\frac{1}{4}$-inch solid steel rib woven in every $7\frac{1}{2}$ inches, shall be applied to the under side of the flat iron bars, with the ribs crossing them at right angles. The ribs in the lathing shall be securely laced to the bars, at every intersection, with No. 18 galvanized wire. All ceilings shall be finished ready for plaster.

111. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

112. The usual practice is simply to apply plaster to the wire lathing around the columns and girders. Filling the space between the iron member and the wire lathing secures a vastly superior protection and involves only a slight additional expense.

The importance of protecting the essential members of the steel framework in an efficient manner should appeal strongly to the intelligent architect, structural engineer, and owner.

SYSTEM A—TYPE 4

113. Floors.—The fireproof floors between the steel beams shall be the Roebling System A, Type 4, construction. T-shaped steel ribs curved to form a segmental arch shall be set at right angles to the beams, parallel to each other, and 24 inches apart. Steel spacers ¼ inch by ¼ inch shall be set, at intervals of 4 feet or less, over the webs of the ribs to hold them rigidly in position. Over these ribs and resting on the flanges of the T's shall be laid painted wire lath with a stiffening rib woven in every 7½ inches, the wire being laced securely to the T-iron ribs with No. 18 galvanized lacing wire. On this T-iron and wire centering, fill cinder concrete mixed in the proportion of 1 part of high-grade Portland cement to 2½ parts of sharp sand and 6 parts of clean cinder or boiler ashes, making the thickness of the concrete at the crown of the arch not less than 4 inches; the concrete shall finish generally 2 inches over the top flanges of the beams.

The flooring shall be subject to test at any point that may be designated by the architect or structural engineer. It shall, in all cases, develop a strength, in 30 days, equal to the capacity of the supporting steelwork.

114. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

SYSTEM B—TYPE 1

115. Floors.—The fireproofing between the steel beams shall be the Roebling System B, Type 1, or flat construction, consisting of a light steel framework embedded in concrete and spanning the interval between the beams in the form of a slab.

The light steel framework shall consist of flat steel bars set on edge with the ends of the bars supported by the iron beams and secured to them. At the under side of the steel bars erect suitable temporary wood centering, or apply permanent wire centering consisting of No. 22 wire lath with a solid steel stiffening rib woven in every 7½ inches, which shall run crosswise under the bars and shall be laced to them, at every intersection, with No. 18 galvanized wire. On the centering so provided, fill cinder concrete, consisting of high-grade Portland cement, sharp sand, and clean steam ashes, to a depth of not less than 3½ inches. All floor beams projecting above or below the concrete floor shall have the webs thoroughly protected by concrete.

116. Ceilings.—The ceilings over the several stories shall be flat. Flat, steel, ceiling bars set on edge shall be supported transversely from the under side of the I beams by suitable iron clamps or

hangers. Painted wire lathing, stiffened with a solid steel rib woven in every 7½ inches, shall be applied to the bars so that the ribs shall cross them at right angles. The ribs in the lathing shall be securely laced to the supporting bars, at every intersection, with No. 18 galvanized wire. All wire ceilings shall be finished ready for plaster.

117. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

SYSTEM B—TYPE 2

118. Floors.—The specification for floors is the same as for System B, Type 1.

119. Ceilings.—All floor beams projecting below the concrete floor shall be incased by suitable temporary wood centering, or permanent wire centering consisting of painted wire lathing stiffened with a solid steel rib woven in every 7½ inches, the lathing to be rigidly supported by suitable light steel furring. The space between the beams and the centering shall be filled solid with concrete. The under surface of the concrete flooring and beams shall be left ready for plaster.

120. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

SYSTEM B—TYPE 3

121. Floors.—The specification for floors is the same as for System B, Type 1.

122. Ceilings.—The floor beams projecting below the concrete floor shall be incased by suitable temporary wood centering, or permanent wire centering consisting of painted wire lathing stiffened with solid steel ribs woven in every 7½ inches, the lathing to be rigidly supported by light steel furring, so as to offset the wire from the beam. The space between the girder and the centering is to be filled solid with concrete. Provide and set in position the flat steel bars for the flat-ceiling construction. Painted wire lathing, stiffened with solid steel ribs woven in every 7½ inches, shall be applied to the under side of the flat steel bars, with the ribs crossing at right angles. The ribs in the lathing shall be securely laced to the bars, at every intersection, with No. 18 galvanized wire. All ceilings are to be finished ready for plaster.

123. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

SYSTEM B—TYPE 4

124. Floors.—The specification for floors is the same as for System B, Type 1.

125. Ceilings.—The under side of the concrete shall provide a continuous plastering surface for flat ceilings, below the under side of the floor beams.

126. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

SYSTEM B—TYPE 5

127. Floors.—The specification for floors is the same as for System B, Type 1, except that the thickness of the concrete slab is to be made 5½ inches instead of 3½ inches.

128. Ceilings.—The specification for ceilings is the same as for System B, Type 2.

129. Columns and Girders.—The specification for columns and girders is the same as for System A, Type 1.

130. When wood centering is employed in erecting Types 2, 4, and 5, and a first-class plaster finish is desired, two coats (a brown coat and a finishing coat) averaging not less than $\frac{5}{8}$ to $\frac{3}{4}$ inch in thickness should be specified under the plaster work. If the brown coat in contact with the concrete is less than $\frac{5}{8}$ inch in thickness, the plaster is liable to become discolored.

131. In cases where the floor beams are more than 8 feet apart and it is desired to lay a cement or granolithic finish on the concrete floor, if temporary wood centering is employed the centers should be "eased" so as to allow the concrete floor to deflect to its permanent position, before the finished surface is laid.

SPECIFICATION FOR A SOLID FIREPROOF PARTITION FINISHING 4 INCHES THICK

132. The studs shall consist of 2″ × 1″ × $\frac{1}{8}$″ channels, spaced 18 inches center to center, and securely fastened at the top and bottom. (Rough wooden frames for all openings will be furnished and set in position under the carpenter's contract.) 2″ × 2″ × $\frac{1}{8}$″ angle irons, or 2″ × 1″ × $\frac{1}{8}$″ channels, shall be secured to the vertical sides of all door

frames and extend from floor to ceiling. After all the light ironwork and wooden frames are in position, painted wire lathing, stiffened with $\frac{1}{4}$-inch solid steel ribs woven in every $7\frac{1}{2}$ inches, shall be laced on both sides of the studs with No. 18 galvanized wire, the stiffening ribs running crosswise over the studs. The space between the wire lathing surfaces shall be filled solid with cinder concrete consisting of 1 part of high-grade Portland cement, $2\frac{1}{2}$ parts of sand, and 6 parts of steam ashes, embedding the channels and forming a solid concrete slab about $2\frac{1}{2}$ inches in thickness.

All surfaces shall be left ready for plaster.

SPECIFICATION FOR A HOLLOW PARTITION FINISHING 4 INCHES THICK

133. The studs shall consist of $2'' \times \frac{1}{8}''$ flat iron or soft steel, spaced 18 inches center to center, and securely fastened at the top and bottom. (Rough wooden frames for all openings and all necessary furring for base board, chair rail, and picture molding will be furnished and set in position under the carpenter's contract.) $2'' \times 2'' \times \frac{1}{8}''$ angle irons, or $2'' \times 1'' \times \frac{1}{8}''$ channels, shall be secured to the vertical sides of all wooden door frames and extend from floor to ceiling. After all the light ironwork, wooden frames for openings, and furrings are in position, painted wire lathing, stiffened with $\frac{1}{4}$-inch solid steel ribs woven in at intervals of $7\frac{1}{2}$ inches, shall be laced to both sides of the studs with No. 18 galvanized wire, the stiffening ribs running crosswise over the studs.

All surfaces shall be left ready for plaster.

SPECIFICATION FOR A SOLID PLASTER PARTITION FINISHING 2 INCHES THICK

134. The studs shall consist of $\frac{3}{4}$-inch channels or $1'' \times \frac{3}{16}''$ flat iron, spaced 16 inches center to center, and securely fastened at the top and bottom. All openings shall be framed with $1'' \times 1'' \times \frac{3}{16}''$ angle iron or $\frac{3}{4}$-inch channels, the vertical members at the door openings to extend from floor to ceiling. All angle-iron framing shall be suitably punched to permit the fastening of the woodwork. When all the light ironwork is in position, painted wire lathing stiffened with a $\frac{1}{4}$-inch solid steel rib woven in every $7\frac{1}{2}$ inches, is to be laced to one side with No. 18 galvanized wire, the stiffening ribs running crosswise over the studs. (All necessary wood furrings for base board, chair rail, and picture molding are to be set in position under the carpenter's contract before plaster is applied.)

SPECIFICATIONS FOR THE STRUCTURAL STEEL-WORK OF A BUILDING

135. The specifications and accompanying plans are intended to cover all the structural steelwork in the building and pertaining thereto, and to provide for a finished piece of work, complete in all its details; the work is to be executed under the supervision of [here is inserted the name of the architect or structural engineer].

GENERAL PROVISIONS AND CONDITIONS

136. The prices given in the accepted proposal shall include the supply and erection in a good, sound, substantial, and workman-like manner, of all items required for the completion of the whole work proposed for, and shall include all the items shown on the drawings and described in these specifications; also, all forms, centering, false works, tramways, machinery, scaffolding, labor, workmanship, tools, and materials necessary and the best adapted to the efficient, prompt, and safe execution of both the permanent and temporary works.

The contractor shall furnish, set, and erect all the steel shown, indicated, or referred to in the accompanying drawings and these specifications, and when the erection is completed, shall remove all materials or devices used in performing the work.

The contractor shall furnish the exact sections, weights, and kind of material called for, shown, or indicated, and shall follow exact details, methods, and instructions called for by the accompanying drawings and these specifications, and such supplemental details and instructions as may hereafter be made, and shall specify with his bid what manufacturer's sections he proposes to use.

The contractor shall submit for approval the general plan of erection proposed to be used, which plan shall be such as will afford the quickest method of erection and the least interference with the work of other contractors.

137. Responsibility of Contractor.—The contractor shall have charge of, and be responsible for, the entire work until completed and accepted by final payment; he shall give his personal supervision to the faithful prosecution of the work, shall not sublet nor assign the same, but shall keep it under his control; he shall also have a competent representative or foreman on the work, who shall receive orders and directions from the architect, structural engineer, or his inspector, and shall have full authority to execute such orders and directions without delay, and to immediately supply materials, tools, and labor as may be required.

The contractor shall cooperate at all times with other contractors and subcontractors of the building, so that when the work is completed it shall be in accordance with the design in every detail.

The contractor for the structural work shall assume all risks and casualties of every description, and will be held responsible for all accidents, negligence, or carelessness pertaining to his work or to the material used, and must make good, without delay, any damage arising therefrom. Every precaution must be used to secure the protection of the workmen and others about the building, and the public on the streets, and the contractor shall indemnify and save harmless the owner from all suits or actions of every name and description brought against him, for or on account of any damages or injuries received or sustained by any party or parties by or from the contractor, his agents, or servants, in the performance of the work.

The contractor will be held responsible for any and all violations and infringements of patent rights and must save harmless the owner from any loss or damage, by suit or otherwise, for any device, process, or materials used in the construction of the work called for in the drawings and these specifications.

The contractor shall be held responsible for any damage that may arise from delays in the execution of his contract or any unnecessary obstruction to the work of other contractors.

The contractor shall be responsible for the observance of all the City Ordinances and Acts of Assembly, and must pay all fines, expenses, etc. that may result from obstructing the street from the time of signing the contract; he shall pay all proper and legal charges to public officers for permits.

The contractor shall maintain at all times a sufficient amount of insurance to protect the owner from loss, in a manner satisfactory to the owner.

138. Drawings.—The drawings illustrating the work consist of (10) sheets of architectural plans, elevations, and details and (8) sheets of structural steel details, plans, and schedules. Said drawings give the scheme, general design, and general details to be followed, and will be supplemented by such additional drawings and details as may be necessary to carry out the general design as here set forth.

Should any figure be omitted from the general plan or from the details, or should any error appear in either, the contractor shall apprise the architect or structural engineer of such omission or error, and shall under no circumstances proceed with the work in uncertainty or without further detail drawings.

Any portion of the work insufficiently expressed or not shown, for the sake of brevity, on the general or detailed drawings, is to be completely executed in the full spirit and meaning.

The contractor shall make and provide all shop, detail, general, and other drawings that may be required for the proper manufacture, execution, and erection of the structural framework of the building, and all that pertains thereto, in harmony with the general and detailed design of the building by the architect or structural engineer.

All such drawings, details, etc. shall be submitted to the architect or structural engineer, and must be approved by him before the work is proceeded with.

When such drawings are submitted for approval, 1 week shall be allowed for examination, after which time they shall be returned, marked approved, or with the changes and corrections indicated. If the drawings are retained for a longer time than 1 week without being either corrected or approved, the contractor shall be allowed an extension of time, equal to the time the drawings are retained beyond the first week.

The approval of any shop drawings shall apply only to the general scheme or arrangement, and will not relieve the contractor of any responsibility for the correctness of all detailed drawings, and for the accurate and complete execution of the work.

No deviation from the approved drawings and these specifications will be allowed, unless written permission shall have been previously granted by the architect or structural engineer.

On the drawings figures shall always have preference over scale, in dimensions.

In case of any doubt or difference of opinion as to the true intent of the drawings and specifications, the architect or structural engineer must be notified at once, and his decision shall be final and binding on both parties to the contract.

139. All shops for the fabrication of structural material prefer to make their own shop drawings. It is not advisable, therefore, for the designer of the building to attempt to make them, but the general and detail drawings or contract drawings should be complete in all respects relating to the weights, areas, fittings, and the requirements of bearing, shearing, number of rivets, etc. The contract drawings should be complete enough to enable the shop to carry out the requirements of the specifications and drawings without further reference to the designer; therefore, all the problems should be worked out on the contract drawings.

140. Workmen.—The contractor shall employ only competent and efficient laborers and first-class mechanics or artisans for every kind of work, and whenever, in the opinion of the architect or structural

engineer, any man is unfit to perform his task, or does his work contrary to directions, or conducts himself improperly, the contractor must discharge him immediately and must not employ him again on the work.

141. Design of Details.—In the design of such members, details, and connections as may hereafter be detailed by the contractor for the structural work, rigid economy consistent with these specifications and good practice will be required.

142. Changes.—The right is reserved to make changes in the details and in the sections required, which may be necessitated by final calculations or changes in arrangement or general design, in accordance, however, with the general scheme as set forth in the accompanying plans and these specifications.

Such changes, however, which may be made, shall be made before the approval of the shop drawings; if required to be made after such approval, allowance will be made as agreed upon or under terms mentioned in the contract.

143. Prints.—Prints of approved drawings shall be furnished by the structural-steel contractor to the general contractor for the use of all subcontractors whose work is affected by the structural work.

Prints of all approved drawings shall also be furnished to the inspector for shop and mill work.

144. Measurements.—All measurements must be verified by the contractor from information to be obtained by him directly, or to be had upon application to the architect or structural engineer, before proceeding with the work.

All measurements, lines, levels, etc. to be made at the site and in the shop shall be made with the most approved instruments for the purpose.

145. Supervision.—The work shall be at all times under the immediate supervision of the architect or structural engineer and of his authorized assistants, who shall have free access to the work at all times and shall be afforded every facilty for inspection.

146. Inspection.—The contractor shall execute his work in the presence of an inspector, during the working hours of the day, unless specially directed otherwise, and shall afford every facility for inspecting the materials and work at all times. Any materials or workmanship deemed of inferior quality, or not in accordance with the finally approved drawings and these specifications, brought to or incorporated in the work, shall be immediately and entirely removed from the vicinity by the contractor, and shall be built anew; and if the directions of the architect or structural engineer are not complied with after written notice, he shall be at liberty to remove the same at the

expense of the contractor, and the cost thereof shall be deducted from any money that may be due him. Materials and workmanship may be reinspected at any time.

147. Time for Completion and Damages for Delay. The proposals must specify the time required by the bidders for the completion of the works from the date of acceptance of the proposal, which time will be inserted in the contract; in case this time is exceeded, the owner is hereby authorized to deduct and retain, out of the payments which may be due or become due the contractor, the sum of—————————dollars as liquidated damages, and not as penalty, for each and every day occupied beyond the time so stipulated.

148. Security.—The contractor will be required to give security, in form to be approved, in the sum of one-half the amount of his contract for the faithful execution of the work.

149. Extra Work.—Whenever, in the opinion of the architect or structural engineer, it shall become necessary to use materials or perform labor that is neither contemplated in the plans of the work nor implied in the specifications referring to said plans, the contractor hereby agrees to furnish such materials and perform such labor as extra work, under such terms as may be agreed upon in writing, without in any way violating the terms of this agreement, and in case terms cannot be agreed upon before the work is undertaken, the contractor shall proceed with such extra work upon the written order of the architect or structural engineer, and the proper value of such extra work shall be determined by three arbitrators, one to be chosen by the owner, one by the contractor, and one to be chosen by the two arbitrators so named, the award of such arbitrators to be binding upon both parties to this agreement.

Should the performance of this extra work require additional time, this time shall be agreed upon in writing, or submitted to the arbitrators previously referred to.

No claims for extra work of any kind will be allowed unless such extra work shall have been ordered in writing by the architect or structural engineer, and the price agreed upon, or unless the work has been so ordered in writing, and the value left for determination by arbitrators under the conditions previously set forth.

150. Monthly Estimates and Payments.—Estimates will be made by the architect or structural engineer on or about the first day of each month, of the value of the work done during the preceding month, and a certificate will be issued to the contractor setting forth the value of this work, with an order on the owner for 80 per cent. of the value of said work, which order shall be payable on or

about the tenth day of the month. The balance shall be retained each month until 60 days after the completion of the work and the acceptance of it by the engineer or architect, subject to all legal or equitable deductions.

GENERAL DESCRIPTION OF THE STRUCTURAL PARTS

151. Character of Structure.—The structure is to be of the skeleton, curtain-beam type, resting on I-beam footings or grillages.

The building is to be (18) stories high, with a basement story and a (flat) (mansard) (pointed) (hipped) roof.

152. Floor Loading.—The floors are to be calculated for a live load of (100) pounds per square foot, and a dead load of (50) pounds per square foot.

The roof is to be calculated for a live load of (50) pounds per square foot, and a dead load of (25) pounds per square foot of horizontal projection.

The partitions are to be calculated as weighing (25) pounds per square foot.

153. Reduction of Live Loads on Girders and Columns.—The floors, beams, and connections are to be proportioned for the full live and dead loads.

The girders are to be proportioned for the full dead load and 85 per cent. of the live load.

The columns are to be figured for the full dead load and 75 per cent. of the live load.

154. If the building is high and all the bending strains due to the wind are provided for, a further reduction in the live load can be made; this matter should be very caretully studied, however, and all the possible strains to which the columns may be subjected provided for before the reduction in live load is made.

155. Floor Beams.—The floor beams are of rolled-steel I beams, attached to girders of similar rolled shapes, or to plate girders as shown on drawings.

The gross floor loads, together with the partition loads where they occur, are to be used in calculating the floor beams and their connections.

In such places as it may be required on account of the difference in level of the bottom flanges of beams where floor arches abut and the use of a raised skewback (hollow-tile arch floor) is not permissible, a small angle shall be riveted to the beams to receive the arch.

156. Roof Beams.—1. *Flat Roof.*—The roof beams to be (10)-inch (25)-pound beams, with tees, angles, etc., as may be required to carry out the drainage system. These roof beams shall be set at varying heights on the tops of the roof girders in order to give the required slope for drainage to the roof surface.

All the structural members shall be arranged to conform to the requirements of the particular system of roof covering specified.

2. *Pointed Roof.*—The trusses are to be made up as detailed on the drawing, the upper chords of two angles (size), or two angles (size), with web-plate (size), as shown.

157. The purlins and subordinate members shall be as shown.

If slates are to be attached directly to the angles or tee purlins, holes at 4-inch centers should be punched.

If the roof is to be covered with porous terra-cotta blocks, the slates can be nailed directly to the terra-cotta blocks.

Trusses shall be attached together with sway bracing of (¾)-inch round rods with turnbuckles, as shown.

Dormer windows are to be framed according to the details shown on the drawing.

3. *Mansard Roof.*—The main roof members (or trusses) shall be made of two angles, or four angles and a web-plate (size) with subordinate members and purlins as shown. Each truss shall be provided with diagonal sway bracing as designated on the drawing.

Purlins for carrying the terra-cotta roofing blocks are to be provided as shown.

Dormer windows shall be framed according to details and if necessary shall be framed in the shop with the use of a wooden templet of the dormer set up in complete form.

158. Floor Girders.—The rolled beams that form the floor girders shall have **hitch-angle** connections to the columns, the shear being transferred directly to the column. Top and bottom bracket angles with two rivets in each flange shall be used in all cases, unless otherwise shown. Where the wind bracing is provided for by a gusset plate under the girders, the bottom bracket angle is to be omitted.

Where plate girders are used as floor girders in place of rolled beams, the same requirements are to govern the detail of the connections, unless otherwise shown.

159. Roof Girders.—Roof girders of rolled beams are to be set with their top flanges on one level, the slope of the roof being allowed for by setting the roof beams on the incline (or if the girders are to be set to the slope of the roof, so specify).

160. If the columns are of one length and the girders are set on one level, the appearance of the rooms is better than if the girders are set to the slope of the roof—if no loft is provided. If a loft or attic is required for distributing pipes, etc., as is usually the case in office-building construction, or for insulation from the heat of the sun, the roof beams can be arranged in the most convenient way, which generally will be the manner herein specified.

161. Curtain Girders.—The curtain girders are to be of rolled sections where the architectural design permits; in all other places plate girders are to be used.

The curtain girders and beams of rolled shapes are to be of the sizes shown in schedule. Separators are to be used at the ends of all abutting beams and under all concentrated loads, etc., and are not to be more than (5) feet apart, as shown. The distances from center to center and the positions with relation to column centers are to be as shown. Top and bottom connection angles shall have not less than two rivets in each flange, and shelf angles shall be placed where shown.

The bottom flanges of the curtain beams are generally on a level with the bottom of the floor beams (with the exceptions shown).

Curtain beams and girders of built-up sections are used in places where two rolled beams cannot be used; these sections are detailed, or indicated on the drawings. Rivet spacing is to be as determined by the allowable rivet bearing and shearing stresses.

162. Separators.—Cast-iron separators are to be used, with the edges conforming to the profile of the beams; beams 12 inches or more in depth shall have two bolts in each separator.

Cast-iron separators are to be used only in footing beams, etc. Wrought-iron separators, consisting of an I section made up of a plate and four angles riveted, shall be used in all other parts of the work requiring separators. These separators shall be set vertically between all double beams, with sufficient rivets to transmit the assumed load or shear—from beam to beam—in such places as may be required. Where the rivets are in tension, bolts shall be substituted.

163. Tie-Rods.—Tie-rods of ($\frac{3}{4}$)-inch round iron (with upset end) with two nuts on each end shall be placed between all floor beams, and floor beams and girders to take up the thrust of floor arches. These tie-rods shall be placed at distances not exceeding 4 feet apart, or as may be directed. Tie-rods shall be placed directly above the bottom flanges of the beams and one nut shall be placed on each side of the web.

164. Columns.—Columns in the exterior walls are to be designed to receive the entire wall load; a reduction of 25 per cent. of total live load is to be made in determining the sectional area required. Connections to floor girders are to be proportioned, however, for 85 per cent. of the total live load.

The columns are to be built up (of open sections consisting of a central web and angles forming an I section, rolled beam, or built up with cover-plates of plates and angles, or channels, as indicated, or of Z bars, as indicated; or of two channels and two plates, as indicated; or of plates and angles).

165. Closed columns of any sections are objectionable because they permit the accumulation of water, chippings, rivet heads, and other débris, which permit the retention of water during erection and subsequently cause rust.

Tie-plates shall be placed at the tops and bottoms of all columns, and all columns having heavy eccentric loads on the side of the weaker axis shall be single-latticed.

At the floors where the columns are spliced, connections shall be provided for the attachment of diagonal lateral tension rods in the floors.

The columns are calculated for eccentricity of loading, and allowance made for same, the point of application of the load being assumed to be 1 inch outside of the column, or otherwise if shown.

The bases of the columns are to be bolted to the footings with through bolts of sufficient length to allow for washer plates and nuts.

The length of the columns below first-floor line varies with the depth of foundations, etc.

Where shown or indicated, single beams shall be run into and attached to central webs of columns to reduce the eccentricity of loading.

QUALITY OF STRUCTURAL STEEL

166. Chemical Composition.—All steel must be made by the open-hearth process (if made by the acid process it shall contain not more than .08 per cent. of phosphorus, and if made by the basic process it shall not contain more than .05 per cent. of phosphorus and it must be uniform in character for each specified kind).

167. Finish.—The finished product shall be perfect in all its parts and free from irregularities and surface imperfections of all kinds. All steel must be free from piping.

168. Requirements.—The tensile strength, limit of elasticity, and ductility shall be determined from a standard test piece, cut from the finished material, of at least ½-inch square section. All broken samples must show a silky fracture of uniform color.

Material that is to be used without annealing or further treatment, is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

Steel shall be of three grades—rivet, soft, and medium.

Rivet steel shall have: Ultimate strength, 48,000 to 58,000 pounds per square inch; elastic limit, not less than one-half the ultimate strength; elongation, 26 per cent.; bending test, 180° flat on itself, without fracture on outside of bent portion.

Soft steel shall have: Ultimate strength, 52,000 to 62,000 pounds per square inch; elastic limit, not less than one-half the ultimate strength; elongation, 25 per cent.; bending test, 180° flat on itself, without fracture on outside of bent portion.

Medium steel shall have: Ultimate strength, 60,000 to 70,000 pounds per square inch; elastic limit, not less than one-half the ultimate strength; elongation, 22 per cent.; bending test, 180° to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

Full size of steel eyebars shall be required to show not less than 10 per cent. elongation in the body of the bar, and a tensile strength of not more than 5,000 pounds below the minimum tensile strength required in specimen tests of the grade of steel from which the bars are rolled. The bars will be required to break in the body, but should a bar break in the head, and yet develop 10 per cent. elongation and the ultimate strength specified, the entire lot shall not be rejected if less than one-third of the bars tested break in this manner.

Pins made of any of the above-mentioned grades of steel shall, on specimen-test pieces cut from finished material, fill the requirements of the grade of steel from which they are rolled, excepting the elongation, which shall be decreased 5 per cent. from that specified.

Punched rivet holes, pitched two diameters from a sheared edge, must stand **drifting** until the diameter is one-third larger than the original hole, without cracking the metal.

The slabs for rolling plates shall be hammered or rolled from **ingots** of at least twice their cross-section.

Pins up to 7 inches diameter shall be rolled. The **blooms** to be used for this purpose shall have at least three times the sectional area of the finished pins.

Pins exceeding that diameter shall be forged under a steel hammer striking a blow of at least 5 tons.

The requirements for elastic limit, elongation, and reduction of area are minimum, and no steel will be accepted that fails to meet these requirements, except as provided for in a duplicate test.

Duplicate tests may be made when the sample tested fulfils five of the six requirements. If the second test and also the average of both tests meet all the requirements, the **melt** may be accepted.

Analyses shall be made showing the amount of phosphorus, carbon, sulphur, silicon, and manganese whenever required, the drillings for these analyses being taken directly from the finished material. All materials used shall be new and of recent manufacture. No materials that have ever been corroded or rusted will be accepted. The material shall in all cases be tested as provided in the following stipulations:

169. Methods of Testing.—The standard test specimen of 8-inch gauged length, shall be used to determine the physical properties of the structural steel. The standard shape of the test specimen for tensile strength for sheared plates shall be as shown in Fig. 1.

For other material, the test specimen may be the same as for sheared plates, or it may be planed or turned parallel throughout its entire length and in all cases, where possible, two opposite sides of the test specimens shall be the rolled surfaces. Rivet rounds and small rolled bars shall be tested of full size as rolled.

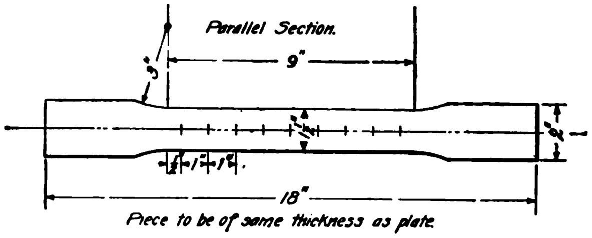

FIG. 1

The material shall be tested on a standard testing machine under the direction and supervision of the inspector, in specimens of at least ½ square inch of section cut from the finished material. Each melt of steel shall be tested, and each section and widely differing gauges of the same sections may be tested; if several sections are rolled from the same melt of steel all of them may be tested at the discretion of the inspector.

Test specimens for bending shall be taken from the finished material of each melt as it comes from the rolls; for material ¾ inch and less in thickness the specimen shall have the natural rolled surface on two opposite sides. The bending-test specimen shall be 1½ inches wide, if possible, and for material more than ¾ inch thick the bending-test specimen shall be ½ inch thick. The bending test may be made by pressure or by blows. Sheared edges of bending-test specimens may be either milled or planed.

Material that is to be used without annealing or further treatment shall be tested for tensile strength in the condition in which it comes from the rolls. Where it is practicable to secure a test specimen from

the material that has been annealed or otherwise treated before use, a full-sized section of tensile-test specimen length, shall be similarly treated before cutting the tensile-test specimen therefrom. For the purpose of this specification, the yield point shall be determined by the careful observation of the drop of the beam or halt in the gauge of the testing machine.

In order to determine whether the material conforms to the chemical limitations prescribed in Art. **166** herein, analysis shall be made of drillings taken from a small test ingot.

170. Variation in Weight.—The variation in cross-section or weight of more than $2\frac{1}{2}$ per cent. from that specified will be sufficient cause for rejection, except in the case of sheared plates, which will be covered by the following permissible variations:

Plates $12\frac{1}{2}$ pounds per square foot, or heavier, when ordered to weight, shall not average more than $2\frac{1}{2}$ per cent. variation above or below the theoretical weight. Plates 100 inches wide, not over 5 per cent. variation either way.

Plates under $12\frac{1}{2}$ pounds per square foot, when ordered to weight, shall not average a greater variation than the following: Up to 75 inches wide, $2\frac{1}{2}$ per cent. above or below the theoretical weight; 75 inches and over, 5 per cent. above or below the theoretical weight.

For all plates ordered to gauge, there will be permitted an average excess of weight over that corresponding to the dimensions on the order equal in amount to that specified in Table I.

TABLE I

Thickness of Plate Inch	For Plates 75 Inches Wide or Under Percentage	For Plates From 75 to 100 Inches Wide Percentage	For Plates Over 100 Inches Percentage
$\frac{1}{4}$	10	14	18
$\frac{5}{16}$	8	12	16
$\frac{3}{8}$	7	10	13
$\frac{7}{16}$	6	8	10
$\frac{1}{2}$	5	7	9
$\frac{9}{16}$	$4\frac{1}{2}$	$6\frac{1}{2}$	$8\frac{1}{2}$
$\frac{5}{8}$	4	6	8
Over $\frac{5}{8}$	$3\frac{1}{2}$	5	$6\frac{1}{2}$

171. Branding.—Every finished piece of steel shall be stamped with the melt number; steel for pins shall have it stamped on the ends. Rivets and lacing steel, and small pieces for tin plates and stiffeners, may be shipped in bundles, securely wired together, with the melt number on a metal tag attached.

QUALITY OF STRUCTURAL CAST STEEL

172. Steel castings shall be made of open-hearth steel, containing from .25 to .40 per cent. carbon, and not over .08 per cent. of phosphorus, and shall be practically free from blowholes.

QUALITY OF STRUCTURAL CAST IRON

173. Except where **chilled iron** is specified, all castings shall be of tough, gray iron, free from injurious cold shuts or blowholes, true to pattern, and of a workmanlike finish. Test bars 1 inch square loaded in middle between supports 12 inches apart shall bear 2,500 pounds or over and deflect .15 inch before rupture.

PROPORTION OF PARTS

174. Least Thickness of Material.—For main members and their connections no material shall be used of less than $\frac{5}{16}$ inch in thickness, and for laterals and their connections no material shall be used of less than $\frac{1}{4}$ inch in thickness. Material for lining and filling vacant spaces shall be of the most convenient thickness.

175. Permissible Tensile Stress.—All parts of the structure shall be so proportioned that the sum of the maximum loads shall not cause the tensile stress to exceed the following values for the several grades of materials: On soft steel, 15,000 pounds per square inch; on medium steel, 17,000 pounds per square inch. Values 20 per cent. higher than these allowable unit stresses may be used for members subjected to stress by wind pressure.

Net sections must be used in all cases in calculating tension members, and in deducting rivet holes they must be taken $\frac{1}{8}$ inch larger than the size of the rivets.

Pin-connected, riveted, tension members shall have a net section through the pin 25 per cent. in excess of the net section of the body of the member. The net section directly back of the pinhole shall be at least 75 per cent. of the net section through the pinhole.

176. Permissible Compressive Stress.—For compression members with flat ends, the allowable unit compressive stresses of 15,000 and 17,000 pounds per square inch, respectively, for soft and medium steel shall be reduced in proportion to the ratio of the length to the least radius of gyration of the section by the following formulas:

For soft steel,

$$P = \frac{15{,}000}{1 + \frac{l^2}{13{,}500\, r^2}} \qquad (1)$$

For medium steel,

$$P = \frac{17{,}000}{1 + \dfrac{l^2}{11{,}000\, r^2}} \qquad (2)$$

where P = permissible working stress, per square inch, in compression;
l = length of piece, in inches, center to center of connection;
r = least radius of gyration of the section, in inches.

177. If there are members in the structure having fixed ends, or struts having pin-connected ends, the formulas for these should be introduced, and, if thought advisable, tables giving the safe allowable compressive stress for different values of $\frac{l}{r}$ should be inserted. It will be noted that while the formulas given have the same form as the Gordon or Rankin formulas, the values are somewhat changed in the specification. It is quite usual for the engineers to stipulate different values in these formulas, as they are individually influenced by the experience and the conditions that obtain.

No compressive member, however, shall have a length exceeding 100 times its least radius of gyration, excepting those for wind bracing which may have a length not exceeding 120 times the least radius of gyration.

178. Combined Compressive Stress of Dead, Live, and Wind Loads.—The maximum unit compressive stress given in Art. **176** may be increased, by vertical loads due to wind, to 19,000 pounds per square inch on soft steel, and 21,000 pounds per square inch on medium steel, reduced in proportion to the ratio of the length to the least radius of gyration, as given in the preceding formulas. The permissible stresses for the connections shall be increased proportionately.

179. Compressive Stress of Dead and Live Loads, Combined With Bending Stress Due to Wind Loads. In proportioning columns, struts, girders, etc. to resist the bending stress due to wind pressure, the section shall be such that the combined dead and live loads, in addition to the bending stress caused by the wind load, shall not produce stresses greater than 19,000 pounds in soft steel and 21,000 pounds in medium steel.

180. Wind Bracing.—Where diagonal rods are not provided to take up the horizontal shear due to wind loads, the connection of

the column to the girder shall be so designed by the use of brackets, portals, or otherwise, as to transfer the resulting moment to the column and girder.

Splice plates securing the columns together shall have sufficient section to transfer the lateral shear and any existing moment at the point specified.

181. Wind Pressure.—In calculating wind pressure, the wind shall be considered as blowing in a horizontal direction at a pressure of 30 pounds per square foot of vertical elevation of the building, if the building is to be erected in an open space. If the building is to be erected in a sheltered location, a reduction in this pressure may be made, but no value less than the following may be used:

At the tenth story: 25 pounds per square foot, with an additional 2½ pounds per square foot for each succeeding story above the tenth, until a maximum of 35 pounds per square foot is reached at the fourteenth story, which pressure shall be used in all stories above the fourteenth.

Below the tenth story: The pressure shall be decreased 2½ pounds per square foot for each succeeding story below the tenth until zero is reached.

182. Alternate Stresses.—Any part of the structure subject to alternate stresses of tension and compression, shall be so proportioned that the total sectional area is equal to the sum of the areas required for each stress. Should the stress be reversed in any possible case, proper provision must be made for such stress in the opposite direction.

183. Transverse Loading of Tension or Compression Members.—Where a floor or roof system rests directly on the top or bottom chord of a truss, the upper or lower chord must be so proportioned that the algebraic sum of the stresses per square inch on the outer fiber, resulting from the direct compression or tension, and three-fourths of the maximum bending moment (the chord being considered as a beam of one panel length supported at the ends) shall not exceed the limiting stresses for tension and compression given in Arts. **175** and **176**.

184. Shearing and Bearing Stresses.—The shearing stress on rivets, bolts, or pins, per square inch of section, shall not exceed 11,000 pounds for soft steel and 12,000 pounds for medium steel; and the pressure upon the bearing surface of the projected **semiintrados** (diameter multiplied by the thickness) of the rivet, bolt, or pinhole, shall not exceed 22,000 pounds per square inch for soft steel and 24,000 pounds for medium steel.

In field riveting, the number of rivets thus found shall be increased 25 per cent. if driven by hand and 10 per cent. if driven by power.

185. Bending Stress on Pins.—The bending stress on the extreme fiber of pins shall not exceed 22,000 pounds per square inch for soft steel, and 25,000 pounds per square inch for medium steel, when the centers of bearings of the strained members are taken as the points of application of the stresses.

186. Plate Girders.—Girders shall be proportioned on the assumption that one-eighth of the gross area of the web is available as flange area. The compression flange shall have the same sectional area as the tension flange; but the unsupported length of flange shall not exceed 16 times its width.

In calculating shearing and bearing stresses on web rivets of plate girders, the whole of the shear acting on the panel next to the abutment is to be considered as being transferred to the flange angles in a distance equal to the depth of the girder.

The shearing stress in web-plates shall not exceed 9,000 pounds per square inch for soft steel, and 10,000 pounds per square inch for medium steel; but no web-plate shall be less than $\frac{5}{16}$ inch in thickness.

The web shall have stiffeners riveted on both sides, with a close bearing against upper and lower flange angles, at the ends and inner edges of bearing plates, and at all points of local and concentrated loads, and also, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, at points throughout the length of the girder, generally not farther apart than the depth of the full web-plate, with a maximum limit of 5 feet.

DETAILS OF CONSTRUCTION

187. Lateral and Sway Bracing.—All lateral and sway bracing subjected to alternate stresses shall be made of shapes that can resist tension as well as compression.

All main members of the roof truss shall preferably be made of two angles, back to back, or two angles and one plate.

Secondary members may be made of one angle, but all other compression members shall be composed of shapes symmetrically disposed.

All nuts on pins used in the construction of any lateral bracing or truss shall be of hexagonal shape.

188. Rivet Spacing.—The pitch of rivets in the direction of the stress shall never exceed 6 inches, nor 16 times the thickness of the thinnest outside plate connected, nor 32 times the thickness at right angles to the stress.

At the ends of compression members the pitch shall not exceed 4 diameters of the rivet, for a length equal to twice the width of the member.

The distance from the edge of any piece to the center of a rivet hole must not be less than 1½ times the diameter of the rivet, nor exceed 8 times the thickness of the plate, and the distance between centers of rivet holes shall not be less than 3 diameters of the rivet.

All joints in riveted work, whether in tension or compression members, must be fully spliced. The sections of compression chords shall be connected at the abutting ends by splices sufficient to hold them truly in position.

All segments of compression members connected by latticing only, shall have tie-plates placed as near the ends as practicable. They shall have a length of not less than the greatest depth or width of the member, and a thickness not less than one-fiftieth of the distance between the rivets connecting them to the compression members.

Single lattice bars shall have a thickness of not less than one-fortieth, and double bars connected by a rivet at the intersection of not less than one-sixtieth of the distance between the rivets connecting them to the member; and their width shall be as follows:

For 15-inch channels, or built sections with 3½- and 4-inch angles.	2½ inches (⅞-inch rivets).
For 12- and 10-inch channels, or built sections with 3-inch angles	2¼ inches (¾-inch rivets).
For 9- and 8-inch channels, or built sections with 2½-inch angles	2 inches (⅝-inch rivets).

All the pinholes shall be reenforced by additional material when necessary, so as not to exceed the allowable pressure on the pins. These reenforcing plates must contain enough rivets to transfer the proportion of pressure that comes on them, and at least one plate on each side shall extend not less than 6 inches beyond the edge of the tie-plate.

189. Plate-Girder Details.—Web-plates of girders must be spliced at all joints by a plate on each side of the web, not less than $\frac{5}{16}$ inch thick, capable of transmitting the full stress through splice rivets.

The flange plates of all girders must be limited in width so as not to extend beyond the outer lines of rivets connecting them with the angles, more than 5 inches, or 8 times the thickness of the first plate. Where two or more plates are used on the flanges, they shall be of equal thickness or shall decrease in thickness outward from the angles.

190. Bracket Angles.—In attaching curtain and floor beams to the columns, the shear shall be provided in the bracket angle, but two top angles shall be provided to prevent movement of the beams with reference to the columns and to insure rigidity of the

joints. At least two rivets shall be put in the top flange of beams, and a greater number when so detailed or required.

191. Column Footings.—Column footings are to be made up of a grillage of rolled sections consisting of channels, I beams, etc. The beams are to be spaced, as shown, with cast separators, having two bolts in each separator.

The beams will be set upon the foundation prepared by the contractor for concrete masonry and shall be leveled up with iron wedges, etc. The footing shall be set true to center lines and truly level. Through bolts, long enough to extend from the bottom of beams to the bottom of column base, with washer plates of proper size, shall be provided and set. Any damage to the concrete caused by the structural contractor shall be repaired at his expense.

192. Connections.—All connections of beams to beams, beams to columns, columns to columns, and other important connections, shall be riveted, wherever the character of the connection will permit. Wherever the use of rivets is not permissible, all holes must be carefully reamed and a turned bolt to a **driven fit** substituted. The use of bolts where rivets can be used or are designed to go will not be allowed without special permission.

193. Connection Angles.—Standard angle connections for beams shall be used under ordinary circumstances where such connections fulfil the conditions for bearing, shearing, etc. Wherever the conditions are unusual or out of the ordinary, connections that fulfil the specifications for bearing, shear, eccentricity, etc. shall be made.

All connections, splices, and framing, when not specially detailed, shall be made in such a manner as to develop the lateral and vertical strength and stiffness of the joint.

194. Coping, Clearance, Etc.—All beams, girders, etc. whose top or bottom flanges are on the same level shall have such flanges coped.

The clearance in all beams of rolled sections shall not exceed ¼ inch, and in any case the clearance shall be confined to the least limit consistent with reasonable erection.

195. Lateral System in Floors.—At the floors where columns are spliced, a diagonal system of rods or flat bars of 1 square inch net section, with **clevises** or other adjustment, shall be provided. These rods or flat bars shall be designed so as to require a minimum thickness in the floor consistent with their purpose. The connection of these laterals to the columns shall be made so as to limit the eccentricity of the stress to the minimum. Flat turnbuckles, clevises, or other approved devices shall be used to adjust these rods.

196. Cutting, Drilling, Etc.—The contractor shall punch, drill, or cut such parts of the work as may be required to receive the work of other contractors, and wherever possible this work shall be shown on shop drawings and done in the shop, and such information shall be furnished by the general contractor, but in such places as it is impracticable to provide for in advance, this work shall be done in the field. In all such field work all holes shall be drilled, and cutting shall be done wherever practicable with a hand saw. No cutting, drilling, etc., on the site will be permitted, however, without the approval of the architect or structural engineer in charge. This clause does not refer, however, to the subsequent fitting up of the stairways by the contractor for stairways, who shall be required to drill stair stringers and carriages to fit his own work.

197. Erection.—The method of erection shall have the approval of the architect or structural engineer.

Temporary bracing shall be provided during the progress of erection if so required.

Wooden hammers shall be used, in place of metal hammers or sledges, in driving home any parts that may fit tightly.

198. Verticality of Columns.—The columns shall be carefully plumbed and kept so until accurately secured in their final position. The outer, or wall, columns shall not deviate from true verticality, measured at right angles to the face of the wall, and the total departure from true verticality shall not exceed 1 inch in any column. Columns in the elevator shaft shall be truly plumbed in all directions.

WORKMANSHIP

199. All workmanship must be of the best kind now in use and where there is any uncertainty as to the quality of the work required by the plans and specifications, the inspector shall require the best class of work that any interpretation will admit.

All plates, angles, and shapes shall, when necessary, be carefully straightened at the shop before assembling. Where the material is of medium steel over $\frac{5}{8}$ inch thick, all sheared edges shall be planed. The several pieces forming one built-up member must fit closely together and when riveted shall be free from twists, bends, or open joints. All portions of the work exposed to view shall be neatly finished.

The ends of all columns and struts shall be milled at right angles to the axis of the column; all other abutting surfaces in compression members shall be truly faced to an even bearing so that there shall be such contact throughout as may be obtained by such means. Abutting members fitted with splice plates must be brought into close and forcible contact and the rivet holes **reamed** into position before

leaving the works, the splice plates being marked so as to be in the same position in erecting. The ends of all the plate girders shall be faced true and square to exact lengths, to insure the verticality of the columns between which they fit. The web-plate stiffeners of plate girders or of built-up girders shall in all cases have a close bearing in the flange angle.

The normal size of the rivets shown on the plans shall be understood to be the actual size of the cold rivet before heating. In all cases, the finished rivet hole shall not be more than $\frac{1}{16}$ inch greater than the diameter of the cold rivet and shall always be of such size that the hot rivet will not drop freely into the hole but will require a slight pressure to force it in place. When pieces forming one built-up member are put together, the holes must be truly opposite; **drifting** to distort the metal will not be allowed. If the holes must be enlarged to admit the rivets, they must be reamed. The rivet heads must be of approved hemispherical shape and of a uniform size for the same size rivet throughout the work. They must be full and neatly finished and concentric with the rivet holes. All rivets, when driven, must completely fill the holes and the heads must be in contact with the surface or countersunk when so required. Wherever possible, all rivets shall be machine driven. Power riveters shall be direct-acting machines worked by steam or hydraulic pressure, or by compressed air.

All reaming of rivet holes must be done after the various pieces have been punched and assembled. After reaming, every hole shall be entirely smooth, showing that the reaming tool has everywhere been in contact with the metal. A reamer shall likewise be used on the outer edge of each hole so as to make a fillet of $\frac{1}{16}$ inch under each rivet head. All holes for field rivets, excepting those in connections for lateral and sway bracing, shall be accurately drilled to an iron templet, or reamed while the connecting parts are in juxtaposition. Holes through medium steel over $\frac{3}{4}$ inch in thickness shall be drilled, or reamed to a diameter of $\frac{1}{8}$ inch larger than the punched holes, so as to remove all the sheared surface of the metal. Where members are connected by bolts that transmit shearing stresses, the holes must be reamed parallel and the bolts turned to a driving fit.

In all field connections, excepting those for lateral and sway bracing, the various parts to be riveted together shall be assembled in the shop and all open holes shall be reamed out while the parts are so assembled, or an iron templet at least 1 inch thick shall be made and all parts reamed to fit.

The heads of eye bars shall be made by upsetting, rolling, or forging into shape. Welds in the body of the bar will not be permitted. The bars must be perfectly straight before boring and the pinholes shall be in the center of the head and on the center line of the bar. The pinholes shall be bored truly parallel with one another and at right angles to the axis of the member, unless otherwise shown

in the drawings, and in pieces not adjustable for length no variation of more than $\frac{1}{64}$ inch for every 20 feet will be allowed in the length between the centers of pinholes. Bars that are to be placed side by side in the structure shall be bored at the same temperature and shall be of equal length, so that upon being piled on each other the pins will pass through the holes at both ends at the same time without driving.

All pins shall be accurately turned to gauge and shall be straight and smooth. The clearance between the pin and pinhole shall be $\frac{1}{32}$ inch for all lateral pins, and for truss pins the clearance shall be $\frac{1}{50}$ inch, for pins $3\frac{1}{2}$ inches in diameter or less. The amount of this clearance shall be gradually increased to $\frac{1}{32}$ inch for pins 6 inches in diameter or over. The pins shall, in all cases, be supplied with steel **pilot nuts** for use during erection.

In general, all material shall be clean and, if necessary, scraped and given one coat of boiled linseed oil after inspection and before shipment. All accessible surfaces shall be given one heavy coat of red lead in raw linseed oil before shipment, and machined surfaces shall be cleaned and given a heavy coat of white lead and tallow after inspection and before shipment.

TESTING FULL-SIZE PARTS

200. Full-sized parts of the structure may be tested at the option of the purchaser; but if tested to destruction, such material shall be paid for at cost, less its scrap value, if it proves satisfactory. If it does not stand the specified test, it will be considered rejected material, and be solely at the cost of the contractor, unless he is not responsible for the design of the work.

INSPECTION

201. All facilities for inspection of materials and workmanship shall be provided by the contractor to competent inspectors, and the architect or structural engineer, or his inspectors, shall be allowed access to any part of the works in which any portion of the material is made. The contractor shall furnish, without charge, such specimens (prepared) of the several kinds of material to be used as may be required to determine their character.

202. Material that is to be used on any work of magnitude or importance should be inspected during its manufacture and fabrication by fully qualified inspectors. The qualifications of an inspector require that he should be a man of integrity and strong character, and also that he should be familiar with the processes of manufacture and fabrication.

As the quality of the material used will depend to a great extent on the selection and judgment of the inspector, he should be chosen and paid accordingly.

If the material is not inspected, that which has been rejected by others may find its way into the work. The cost of inspection, being a special service, should be paid for by the owner.

PAINTING

203. All ironwork before leaving the shop shall be thoroughly cleaned from all loose scale and rust, and given one good coat of pure boiled linseed oil, well worked into all joints and open spaces.

In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

Pieces that are not accessible for painting after erection shall have two coats of paint.

The paint shall be of best quality of red-lead paint mixed with pure linseed oil, or such as may be specified in the contract.

After the structure is erected, the ironwork shall be thoroughly and evenly painted with two additional coats of paint, mixed with pure linseed oil, of such quality and color as may be selected.

Pins, pinholes, screw heads, and other finished surfaces shall be coated with white lead and tallow before being shipped from the shop.

After erection, the floor beams, tie-rods, laterals, etc., all roof beams and girders, and all such parts as have no concrete filling or envelope to protect them from rust, shall be painted with one or two coats of red lead and boiled linseed oil, as may be required.

PROPOSALS AND BIDS

204. Proposals for the structural work will be sent directly to the architect or structural engineer, who will select the bid to be used, which bid the general contractor will be required to incorporate in his bid. Bids must be submitted as follows:

All material shall be furnished and erected complete in every detail, to the satisfaction of the architect or structural engineer, painted with one coat of linseed oil in inaccessible places when put together in the shop, and painted with red lead and oil in accessible places, as specified under *Painting*.

Item 1.—Price per pound of all simple rolled shapes that have no riveted work attached to them, excepting separators and hitch or connection angles, such as floor beams and channels, curtain beams and channels, tie-rods for floor arches, and simple tees and angles. Also,

all rolled sections of beams, channels, etc., forming the column footings, with bolts for same, etc., beams used over vaults, floor laterals if not riveted, etc.

Item 2.—Price per pound of all simple rolled shapes, with angles riveted to their webs or flanges, to receive floor arches, roof arches, etc.

Item 3.—Price per pound of all compound shapes, such as built girders, built columns, splice plates, gusset plates, and other pieces forming parts of any riveted or compound members, and other parts not included in the other classes; floor laterals, if riveted.

Item 4.—Price per pound of cast-iron base plates and other castings necessary for the completion of the structural framework.

No material that does not form part of the finished structure will be paid for.

Item 5.—Price per pound for painting with one and also with two coats of the best red lead and boiled linseed oil or other paint that may be selected, costing no more per gallon. The prices shall cover all the above classes of material after erection. Such painting shall be done on all floor beams, ties, laterals, all roof girders and beams and such other parts as may not be fireproofed or rust-proofed with concrete, and also on all such other parts as may be required or directed, after erection—in order to prevent corrosion, before the application of the concrete or fireproofing.

205. Time of Erection.—Bidders must state in their proposals the earliest date on which they will agree to erect the structure complete, and also state the various dates on which they can complete portions of the work, under such damages as are hereinbefore set forth.

Bids must include a close estimate of the various amounts of materials in each class required in the structure as outlined in the plans and specifications.

GLOSSARY

arris, the line, edge, or hip in which the two straight or curved surfaces of a body, forming an exterior angle, meet; especially the sharp edge between two adjoining channels of a Doric column.

asphaltic mastic, or asphaltic cement, a mixture commonly made of refuse tar from gas houses, mixed with slaked lime and gravel.

bloom, a roughly prepared mass of iron, nearly square in section, and short in proportion to its thickness, intended to be drawn out under the hammer or between the rolls into the bars.

bush-hammered, the finish on stonework made by using the bush hammer, which has a head from 4 to 8 inches long, with ends from 2 to 4 inches square and cut into a number of pyramidal points; a stone finished in this manner has the appearance shown in Fig. 2.

chilled iron, iron that has been run in a chill, or metal mold, in which certain kinds of iron castings, as car wheels, are made. The surfaces in contact with the mold are hardened by sudden chilling.

chink, to fill up the interstices.

clevis, an iron bent in the form of a stirrup, horseshoe, or the letter **U**, with the two ends perforated to receive a pin.

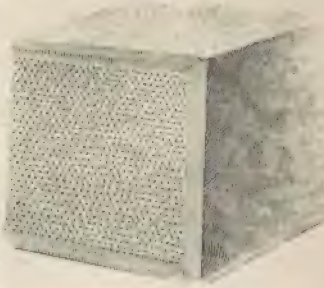

FIG. 2

coffer dam, a water-tight, wooden enclosure built in a body of water, in order to obtain a firm and dry foundation for piers, bridges, etc., by pumping out the water from its interior. It is sometimes formed by driving, close together, two or more rows of piles, which rise above the level of high water, and packing clay between them.

cut-pointed, in this method of pointing the joint is filled with the pointing mortar, which is afterwards cut off so that the joint projects a trifle from the edge of the material.

draft, a line on the surface of a stone hewn to the breadth of the chisel.

drifting 1, in testing, the process of enlarging a hole in a metallic plate by driving into it a drift bolt, which is a round, slightly tapering piece of steel.

drifting 2, driving a drift bolt into two holes that do not come directly in line, thus enlarging them until they are exactly opposite, or until the pin can be inserted.

driven fit, fitting so tightly that it must be driven in place.

fine-pointed, where a smooth finish is not required but it is desired to dress the face of a stone so that it will not project more than ¼ to ½ inch, the rock face is taken off with a point and the surface is said to be rough- or fine-pointed. If the point is used over every inch of the surface, it is *rough-pointed*, and if over every half inch, it is *fine-pointed*.

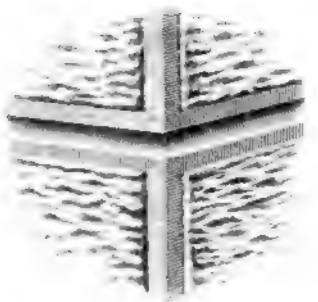
FIG. 3

hard set, cement is said to have attained a hard set when it has become brittle; that is, when it has reached such a degree of hardness that it cannot be altered without causing a fracture.

hitch angle, an angle used in making connections of structural members, such as floor beams and girders, girders and columns, etc. It is generally a short piece cut from a standard angle, but may also be a bent plate or clip.

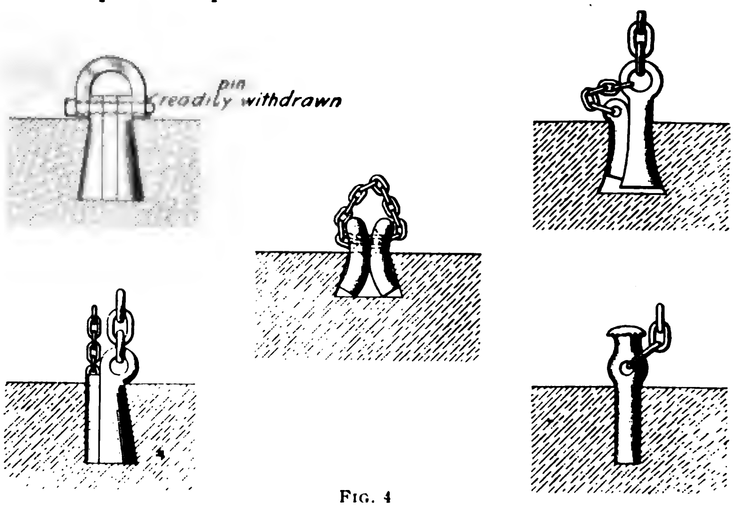

FIG. 4

hollow-pointed, pointed and finished with a hollow joint that is formed by using a convex-edged jointer to press the mortar into the joint. A joint formed in this manner is shown in Fig. 3.

ingot, a mass of metal cast in a mold.

initial set, cement attains its initial set when it loses its plasticity and its virtue is destroyed by any vibration or disturbance.

intrados, the interior or lower line, curve, or surface of an arch or vault.

lewis, a contrivance for securing a hold on a block of stone in order that it may be raised from its position by a derrick. It consists of two side pieces that fit into a dovetail recess cut in the stone, and between which a ring tongue is put and fastened in such a way that when lifted, the lewis secures a firm hold by wedging itself on the dovetail. This is illustrated in Fig. 4.

melt, a charge of metal placed in a cupola or pot for melting. The product of such a charge is also called a melt.

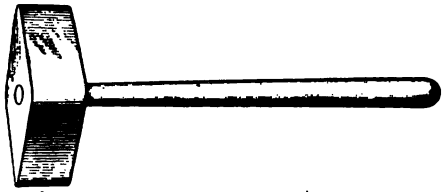

FIG. 5

neat cement, pure cement mixed with water.

peen-hammered, finished with the peen hammer or ax, shown in Fig. 5. It is used after the point on granite and other hard stones to reduce the face to a level.

pilot nut, a tapered nut screwed upon the end of a pin to protect the screw threads when the pin is being put in place and to pilot it through the holes of the connection members. It is afterwards removed and the ordinary nut screwed on. The details of a pilot nut are shown in Fig. 6.

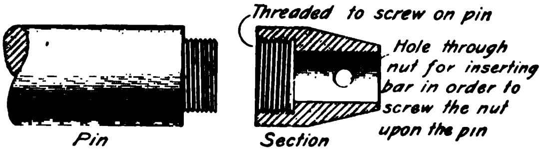

FIG. 6

ream, enlarging holes in metal by cutting out with a rotatory cutter.

rift-sawed, sawed in quarters and then cut into planks of the required size.

riprap, a foundation or parapet of stones thrown together without any attempt at regular structural arrangement, as in deep water or on a soft bottom.

scabbled, dressed with a broad chisel or heavy pointed pick, preparatory to finer dressing.

semiintrados, half intrados; in this connection the surface of the hole is called the intrados.

spandrel, the triangular space comprehended between the outer curve, or extrados, of an arch, a horizontal line drawn through its apex and a vertical line through its springing; also, the wall space between the outer moldings of two arches and a horizontal line, or string-course, above them, or between these outer moldings and the intrados of another arch rising above and enclosing the two.

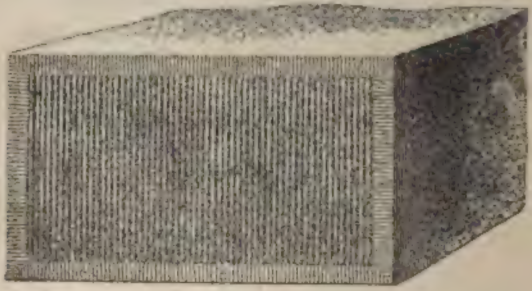

FIG. 7

tooled, finished with a flat chisel from $3\frac{1}{2}$ to $4\frac{1}{2}$ inches wide. The lines are continued across the width of the piece, as shown in Fig. 7.

Vicat needle, a needle used in a test for cement, invented by M. Vicat. The needle test is practiced to determine the time in setting and the relative hardness attained at stated intervals during the process of hardening of the cement samples.

voussoir, a stone, in the shape of a truncated wedge, that forms part of an arch; a ring stone.

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